ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

# Master's Thesis

TATIANA LARIONOVA

## NONLINEAR NUMERICAL EVALUATION OF BAROQUE EPOCH FOUNDATION STRUCTURES ON THE EXAMPLE OF THE BROUMOV GROUP OF CHURCHES FOOTINGS

University of Minho

**Czech Republic** | 2021

SAHC

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

# DIPLOMA THESIS ASSIGNMENT FORM

## I. PERSONAL AND STUDY DATA

Surname: Larionova Name: Tatiana Personal number: 504530

Assigning Department: Departmanet of Mechanics

Study programme: Civil Engineering

Branch of study: Advanced Masters in Sturctural Analysis of Monuments and Historical Construction

## II. DIPLOMA THESIS DATA

Diploma Thesis (DT) title: Nonlinear numerical evaluation of baroque epoch foundation structures on the examples of the Broumov Group Churches footings.

Diploma Thesis title in English: see above

Instructions for writing the thesis:
The thesis should be written in consequent parts. Historical overview of the epoch and the local conditions in the Broumov site. Analysis of the foundations of the Broumov Group of Churches. Namely Saint Jacob Church (in Ruprechtice), St. Ann Church (in Vižňov), All Saint Church (in Heřmánkovice), and the Church of St. Barbara (in Otovice). Description of the overall shape. Analysis of geological profiles and soil and rock parameters estimation. The wide analysis would be focused on the nonlinear calculation of the settlement employing GEO5 FEM software, using analytical elastic layer solution, and comparing results with the codes. In conclusion, analyze and summarize obtained facts, conclusions of the analyses, and recommendations for possible future remediation of substructures.

List of recommended literature:
Terzaghi K., Peck R. B., Mesri G.: Soil Mechanics in Engineering Practice
Šejnoha M.: GEO FEM - Theoretical manual
https://www.finesoftware.eu/user-guides/

Name of Diploma Thesis Supervisor: Pavel Kuklík

DT assignment date: March 17. 2021 DT submission date: July 6, 2021

DT Supervisor's signature Head of Department's signature

## III. ASSIGNMENT RECEIPT

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

Assignment receipt date Student's name

# DECLARATION

Name: Tatiana Larionova

Email: larionova.tatiana.v@gmail.com

Title of the M.Sc. Dissertation: Nonlinear numerical evaluation of baroque epoch foundation structures on the example of the Broumov Group of Churches footings

Supervisor(s): Prof. Ing. Pavel Kuklík

Year: 2020/2021

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

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

University: Czech Technical University in Prague

Date: 06.07.2021

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To my future life

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## ACKNOWLEDGEMENTS

First of all, it should be mentioned that the times when this work was being performed were quite complicated which created additional challenges for its accomplishment. But despite this fact, it was completed in time. Therefore, I wish to express my gratitude to my supervisor Prof. Ing. Pavel Kuklík, who tried to simplify my path and gave it the right direction. Moreover, I want to thank him for the interesting discussions that accompanied our meetings. Additionally, I would like to acknowledge Dr. Kateřina Kovářová for her assistance with the particular part of this work.

Besides, I would like to acknowledge a gratitude to Prof. Paulo B. Lourenço, Prof. Daniel Oliveira and SAHC Consortium for providing support and scholarship that enabled me to join this program after all these years.

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Apart from that, I am obliged to the management of the company for which I am currently employed, as it gave me an opportunity to study parallelly and provided additional financial support during this complicated period.

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## ABSTRACT

The Czech Republic is home to many historical monuments and places, including fairy-tale castles, iconic geomorphological features, and picturesque villages. Many churches and chapels in the countryside were built in the 17th and 18th centuries using a Baroque architectural style to transform the landscape.

Broumov is a unique region in the northeast of the Czech Republic, that is distinguished not only by its natural conditions and beauty but also by its rich cultural heritage. There are several dozens of architecturally valuable cultural monuments, that complete the typical landscape of this area. One of remarkable monuments is the complex of churches, that were built in a short period from 1709 to 1743 by famous bohemian architects father and son Dientzenhofer. This complex due to its uniqueness is known as the Broumov group of churches.

The main goal of this dissertation was to evaluate the knowledge and skills of ancient masons on the example of four churches in the Broumov group, with a focus on foundation structures. In addition, there is a certain specificity in an estimation of the impact of funding and the amount of money on the quality of the construction work.

After a historical overview and presentation of the geological conditions of the region, the work was focused on the calculation of the depth of influence zones in the subsoil, which mainly affects the non-linear character of settlement. Several methods were used to determine them. Starting with the analytical method by means of DEPTH software, then GEO5 FEM software was used for the numerical solution. Additionally, the same calculations were also performed according to the Russian standard using Foundation software. Thus, the methods were not only compared but verified as well. Input parameters describing soil properties were taken based on visual inspections and recommendations of the Department of Geotechnics CTU in Prague.

The results obtained using these different methods were used to compare the quality of the foundation structures of particular churches. Finally, some of the possible causes of damage to building materials, especially stone, were analysed. Recommendations and some solutions were proposed for the case of remediation.

**Keywords:** Baroque, soil-structure interaction, foundation settlement, cultural heritage

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## ABSTRAKT

Česká republika je domovem mnoha historických památek a míst, včetně pohádkových hradů, ikonických geografických prvků a malebných vesniček. Mnoho kostelů a kaplí na venkově bylo postaveno v 17. a 18. století barokním architektonickým stylem, který proměnil krajinu.

Broumov je jedinečný region na severovýchodě České republiky, který se vyznačuje nejen přírodními podmínkami a krásou, ale také bohatým kulturním dědictvím. Nachází se zde několik desítek architektonicky cenných kulturních památek, které dotvářejí krajinu této oblasti. Pozoruhodný je komplex kostelů, které byly postaveny v krátkém období od roku 1709 do roku 1743 otcem a synem Dientzenhoferem. Tyto kostely, díky své jedinečnosti, jsou známé jako Broumovská skupina kostelů.

Hlavním cílem diplomové práce bylo zhodnotit znalosti a dovednosti starověkých zedníků na příkladu čtyř kostelů z Broumovské skupiny. Práce byla zaměřena zejména na základové konstrukce. Určitou specifičností je navíc odhad vlivu financí na kvalitu stavebních prací.

Po historickém přehledu a představení geologických podmínek regionu se práce zaměřila na výpočet hloubky deformačních zón v podloží, která zejména ovlivňuje nelineární průběh sedání. K jejich určení bylo použito několik metod. Počínaje analytickou metodou pomocí softwaru DEPTH. Pro numerické řešení byl použit software GEO5 FEM. Stejné výpočty byly také provedeny podle ruského standardu pomocí softwaru Foundation. Toto posloužilo nejen k porovnání uvedených metod, ale k jejich určité verifikaci. Vstupní parametry popisující vlastnosti půdy byly převzaty na základě vizuálních prohlídek a doporučení katedry geotechniky Fakulty stavební ČVUT v Praze.

Výsledky získané pomocí těchto různých metod se využily k porovnání kvality základových konstrukcí dílčích kostelů. Nakonec byly analyzovány některé z možných příčin poškození stavebních materiálů, zejména kamene. Pro případ sanace byla navržena doporučení a některá řešení.

**Klíčová slova**: Baroko, interakce stavby s podložím, sedání základů, kulturní dědictví

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## РЕЗЮМЕ

Чешская Республика является домом для большого количества исторических памятников и локаций, в том числе сказочных замков, знаковых геоморфологических объектов и живописных деревень. Многие церкви и часовни в сельской местности были построены в 17-18 веках в архитектурном стиле барокко, который преобразил значительно ландшафт.

Броумов - уникальный регион на северо-востоке Чешской Республики, который отличается не только своими природными условиями и красотой, но и богатым культурным наследием. Здесь расположено несколько десятков архитектурно ценных памятников культуры, которые дополняют типичный ландшафт этой местности. Примечателен комплекс церквей, которые были построены за короткий период с 1709 по 1743 год известными богемскими архитекторами отцом и сыном Динценгоферами. Эти церкви в силу свой уникальности известны как Броумовская группа церквей.

Основной целью дипломной работы является оценка знаний и умений древних каменщиков на примере четырех церквей Броумовской группы с акцентом на фундаментные конструкции. Кроме того, есть еще одна особенность работы -это оценка влияния финансирования и количества денег на качество строительных работ.

После исторического обзора и презентации геологических условий региона, работа сосредоточена на расчете глубины сжимаемой толщи грунта, которая в основном влияет на нелинейный характер осадки. Для ее определения использовалось несколько методов. Начиная с аналитического метода с помощью программного обеспечения DEPTH, а затем для численного решения использовалась программа GEO5 FEM. Кроме того, такие же расчеты были выполнены согласно российскому своду правил с использованием программного обеспечения Foundation. Это послужило поводом не только для сравнения этих методов, но и для их, в определенном смысле, проверки. Входные параметры, описывающие свойства грунтов, были приняты на основе визуальных осмотров и рекомендаций кафедры геотехники, факультета гражданского строительства Чешского технического университета в Праге.

Результаты, полученные посредством этих различных методов, были использованы для сравнения качества фундаментных конструкций выбранных церквей. В заключении, возможные причины причины повреждения строительных материалов, особенно камня были проанализированы. А также, были предложены рекомендации и некоторые решения в случае восстановления.


**Ключевые слова**: барокко, взаимодействие сооружения с грунтом, осадка фундамента, культурное наследие.

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# 1. INTRODUCTION

The Czech Republic is located geographically in close vicinity to the middle of Europe and is surrounded by different countries. Due to that, the Czech culture was formed under influence of its neighbours, political and social changes, wars and peacetimes. Prague’s importance as a cultural center had been changing throughout history, but the Czech culture remains distinctive up to the present day.

The Czech Republic is well-known for its fascinating history and rich cultural heritage. The country abundant in historical cities and villages with more than 1500 castles and 14 UNESCO protected heritage sites, including fairy-tale castles and chateaux.

Broumov is a picturesque region, where human husbandry and diverse natural environment have been interacting for centuries, resulting in aesthetically and ecologically valuable landscape. It is well-known for its extensive sandstone rock cities and unique cultural heritage, which comprises especially religious and folk architecture. The Baroque period marks the character of the Broumov region to this day. Under the abbots Tomáš Sartorius (1663-1700) and Otmar Daniel Zinke (1700-1738), a world-unique architectural sacral activity was carried out.

Broumov group of churches is a unique complex of ten baroque churches that were built in the eighteenth century on the territory of the Broumov monastic estate by famous bohemian architects - father and son Dientzenhofer. The churches differ from each other in size, style and shape but altogether they create an amazing architectonic complex. They are located in small villages around Broumov town.

The main aim of this dissertation work is to evaluate the knowledge and proficiency of ancient masons as well as how the funding affects the construction works. For this purpose, four churches of Broumov group were selected and analysed using different approaches. First of all, the analytical method was utilized with help of DEPTH software after that the numerical solution was applied using GEO5 software. Furthermore, all foundations of churches were analysed in accordance with the Russian Code which also represent the analytical solution. After all, the results were compared to each other and based on them the quality of foundations were analysed.

Finally, the potential reasons of deterioration of building materials were discussed as well as damages found during the inspections and some general recommendations were proposed.

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# 2. HISTORICAL OVERVIEW

Before proceeding with any in-situ test or performing any kind of calculations of historical structure, it is necessary to conduct a historical survey. It is an important part of assessment of building because it can provide data on used materials, possible construction techniques and alterations that the building may have undergone. Furthermore, with such that information in disposal, it can be possible to estimate potential causes of damage in the structure. Finally, sometimes it is the only source of obtaining data due to inability to carry out tests on-site.

## 2.1 History of Bohemia

Bohemia is a historical region in central Europe occupying the western and central thirds of the Czech Republic. The region has the area of 52,750 km² and it is the place of residence of 6 million people of the country's 10 million population. It is flanked by Germany, Poland, the Czech historical region of Moravia, and Austria, and its border is formed by four mountain ranges, (Bohemia, n.d.).

The Czech history undergone through several periods - the Great Moravian Empire (9$^{th}$ century), the Premyslid Dynasty (9-14$^{th}$ century), the Luxembourg Dynasty (14-15$^{th}$ century), the Hussite Revolution (1419 - 1436), The Jagellon Dynasty (15-16$^{th}$ century), the Habsburg Dynasty (16-20$^{th}$ century), the foundation of the modern Czech nation and an independent state (since 1918).

Bohemia derives its name from a Celtic tribe, the Boii, who inhabit the region during the last few centuries BC. The Slav tribes arrived at the region from the east during the early centuries AD. The most powerful of these tribes were the Cechove, or Czechs, (History of Bohemia, n.d.).

Bohemia was united under the Přemyslid dynasty, under whose helm it also became an autonomous part of the Holy Roman Empire after accepting Christianity in the ninth century. Charles IV, Holy Roman Emperor, of the subsequent Luxembourg dynasty established the first university in Central Europe in Prague and laid down the formula for the region's economic, cultural, and artistic boom. The period of glory was repeated under Emperor Rudolph II Habsburg, who embraced artists and scientists from all over Europe, (History of Bohemia, n.d.).

Through the Hussite Wars in the fifteenth century, Bohemia took a stand for the freedom of religion, drawing on the spiritual strength and martyrdom of Jan Hus. Led by the ingenious Jan Žižka, the under-armed and under-trained, yet disciplined and arduous, peasant armies believed they were invincible as long as they stayed united in faith. For the most part, Bohemia's history was intertwined with that of Germany and Austria, from which it benefited both economically and culturally, while at the same time facing varying degrees of oppression, (Bohemia, n.d.).

The country ceased to exist in 1918, with the formation of the joint state of Czechs and Slovaks: Czechoslovakia (Figure 1). After World War II, Czechoslovakia aligned itself with the communist block, from which it was extricated in 1989. Four years later, Czechs and Slovaks parted amicably, and Bohemia became part of the Czech Republic, (Bohemia, n.d.). Václav Havel was elected as the first president of the Czech Republic.


**Figure 1 - Bohemia within Czechoslovakia in 1928 (Bohemia, n.d.)**

**Figure 2 – Bohemia within the Czech Republic nowadays (in green) (Bohemia, n.d.)**

The Czech constitution dated 1992 refers to the "citizens of the Czech Republic in Bohemia, Moravia and Silesia" and proclaims continuity with the statehood of the Bohemian Crown. Bohemia is not an administrative unit of the Czech Republic; instead, it is divided into the Prague, Central Bohemian, Plzeň, Karlovy Vary, Ústí nad Labem, Liberec, and Hradec Králové Regions, as well as parts of the Pardubice, Vysočina, South Bohemian, and South Moravian Regions, (History of Bohemia, n.d.).

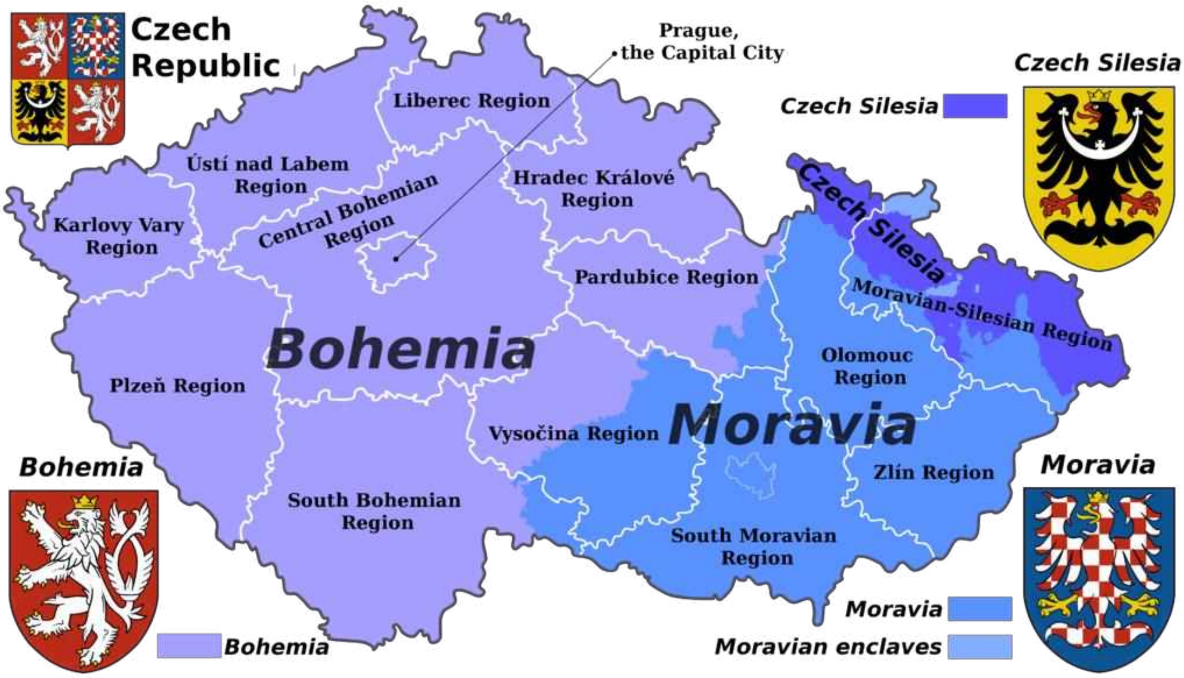


**Figure 3 - Map of the Czech Republic with traditional regions and current administrative regions (Czech lands, n.d.)**

## 2.2 Bohemian baroque architecture

Baroque architecture had developed in Bohemia from the beginning of the 17th century until the second half of the 18th century. Baroque was an exuberant combination of arts, giving origin to highly ornamented facades and interiors, which incorporated painting and sculpture as part of the decoration. Following the return of Catholicism, the church and the nobility had abundant resources, so a lot of the architecture produced during this period was for churches and palaces. One of the opportunities for Baroque architects was the construction of churches in places of pilgrimage, often with adjacent monastic complexes. Churches were often placed in the landscape on a hill to be visible from afar, or in the middle of spaces surrounded by cloisters with chapels, thus creating complete picturesque units. Where the terrain allowed, it was composed mainly in front of the facade of the staircase, which played an important corner in the grand ceremony of the pilgrimage festivities. An extraordinary role is also attributed to the symbolism, which is reflected in the floor plans of buildings related to their dedication, (Horáková, 2010).

The style itself has gone through several developmental stages, which generally date as follows:

- early baroque (1618-1700)
- high baroque (1700-1740)
- late baroque (1740-1780)

Early baroque is characterized by a predominant share of foreign, especially Italian architects (for instance, Carlo Lurago, Giovanni Domenico Orsi de Orsini, Francesco Caratti), (Horáková, 2010).
The Wallenstein Palace (Figure 4) was first baroque palace in Prague. It was built by Italian architect Giovanno Pieroni and Andrea Spezza from 1621 to 1630. The first church in baroque style is considered to be The Church of Our Lady Victorius in Prague. Originally, it was built between 1611-1613 in renaissance style, but in 1644 the façade was completed in baroque style (Figure 4).

**Figure 4 – The Church of Our Lady Victorius, Prague (right) and Wallenstein Palace (left) (Prague.eu, n.d.)**

The most famous architects of the Czech High Baroque style were Christoph Dientzenhofer (in Czech - Kryštof Dientzenhofer) and his son Kilian Ignaz Dientzenhofer (in Czech - Kilián Ignác Dientzenhofer) as well as Jan Blažej Santini-Aichel. Christoph Dientzenhofer and Kilian Iznaz Dientzenhofer were also known for their style called "radical Baroque", which was inspired by examples from northern Italy, particularly by the works of Guarino Guarini, and which seeks to express movement. It is characterized by the curvature of walls and intersection of oval spaces. The Church of St. Nicholas is a superb example of High Baroque architecture; it stands along with the former Jesuit college in the center of the Lesser Town Square (Figure 5). Today's Church of St. Nicholas is one of the most valuable Baroque buildings to the north of the Alps. Construction lasted approximately one hundred years, and three generations of great Baroque architects worked on the church – father and son Dientzenhofers and Anselmo Lurago, (Kostel Sv. Mukulase, n.d.).

The late Baroque style was usual in the Crown of Bohemia during the reign of queen Maria Theresa (1740-1760). At the same time, the Rococo style came to the Czech lands.

**Figure 5 – The St. Nicholas Church (Kostel Sv. Mukulase, n.d.)**

## 2.3 Broumov region

The region of Broumov is geographically and historically a distinctive region with a wild and poetic landscape. For many centuries this region was once a very important cultural center enriched by artistic monuments and edifices, that were built during the administration of the Benedictine monks. A harmonious, aesthetic, and ecological society emerged that made this region quite significant. The landscape of this region is quite diverse; there are mountains, valleys and bizarre rock formations and also mosaics of forestland, meadows, and grassland. The region's unique location with its picturesque landscape, the beautiful architecture found in the churches and folk monuments are only one part of what makes this area exceptional, (The Region of Broumov, 2013).

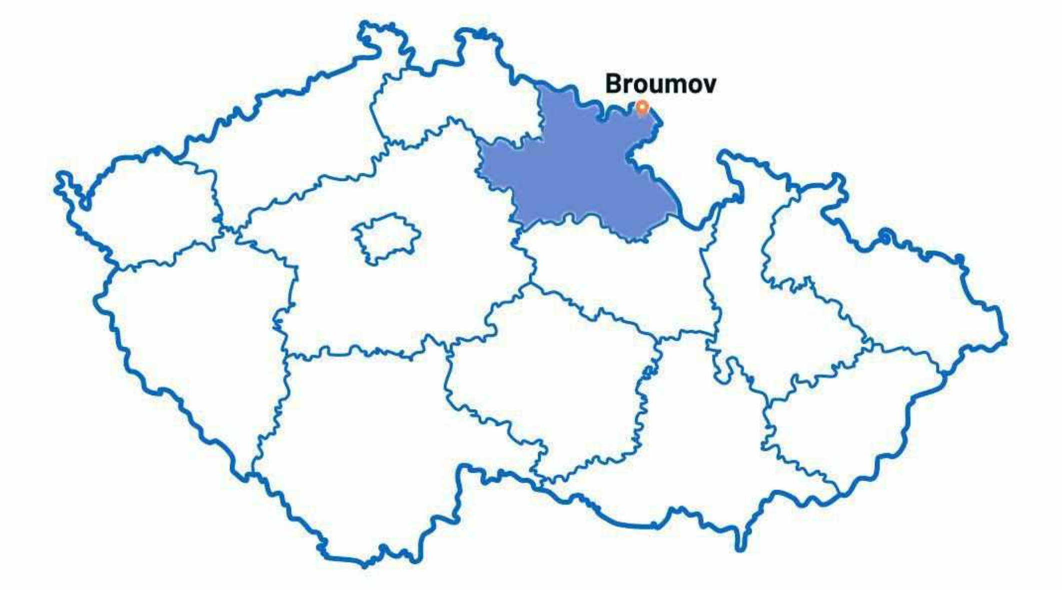


**Figure 6 - The location of Broumov region**

This region is famous for its historical buildings and cultural treasures. In the towns of Broumov and Police nad Metují, there are two Benedictine monasteries. In the village of Křinice there is typical original German folk architecture, there is a group of Baroque churches in Bezděkov, Božanov, Broumov and Heřmánkovice, there are chapels in Hvězda, Martínkovice, Otovice, Ruprechtice, Šonov, Vernéřovice and Vižňov.

This rocky headland was colonized in the 13th century by the Benedictine order of Břevnov when the land was given to them by King Přemysl Otakar I. People who settled in the Police area were mostly Bohemian while those of German origin settled in Broumov. German and Bohemian aristocracy settled in the towns of Stárkov and Teplice; however, these areas were inhabited mostly by Germans lated. After the construction of the monasteries in Broumov and Police, the monks carried out a planned colonization and this led to regular forms of settlements in the form of villages with farmland around waterways. The Benedictine Monks built additional functional buildings in the towns of Broumov and Police nad Metují, (The Region Of Broumov, 2013).

During seven centuries of Benedictine rule, this "border" area witnessed many political and social conflicts - Czechs versus Germans, Catholics versus Protestants, Church versus State. The region of Broumov always managed to ride over many obstacles that were brought about by these conflicts, wars and also by the plague after the Thirty Years War that claimed the lives of many of its citizens. This was mainly possible because of the good economic status and business activities of the Benedictine abbacy.

The development of textile industry and political freedom in the 19th century brought prosperity to the region of Broumov. The textile industry was expanded to other towns and villages and goods were exported outside the region. Another boost for the developing industry was the construction of the railway in Choceň (1875), that connected Vienna with Silesia and Klodzko. During the "golden textile era" the region of Broumov was one of the most populated regions in Bohemia with the density of population of 159 people per square km.

The 20th century brought about painful years. The majority of inhabitants of German origin refused citizenship of Czechoslovakia. The economic crises of the 1930s trigged many demonstrations. The Henlein party was founded in 1933 and after the Munich Agreement the Broumov region was divided - Broumov and Teplice became part of Hitler's Third Reich and Police was part of the Protectorate Bohemia - Moriavia. The breakdown of Broumov's development came after The Second World War. The communist ideology that followed started to destroy the local heritage. Industries and agriculture were nationalized, and socialist villages were built. The township of Broumov was abolished and so the cultural, social, and industrial continuation of this region was interrupted, (The Region Of Broumov, 2013).

## 2.4 Broumov group of churches

The baroque churches, the so-called "Broumov Group", built on the territory of the Broumov monastic estate in accordance with the plans of Christoph Dientzenhofer and his son Kilian Iznaz Dientzenhofer form a unique architectural landmark of the region. The churches were built in a relatively short time, in the period 1709-1743. They are a result of unusual construction and economic boom initiated by abbot Otmar Zinke - a very capable provost of the Benedictine monastery of Břevnov-Broumov. Despite the unquestionable economy of their design, these country churches bear a seal of uniqueness and creative originality worthy of the well-known name of its authors - father and son the Dientzenhofers, (The Broumov Group of Churches, 2013).

Various sources provide different number of churches referred to as Broumov group of churches. For example, some authors say that St. Prokop Church is located beyond the administrative borders of Broumov parish, but according to (The Region Of Broumov, 2013), the following churches are considered to be a part of Broumov group of churches:

1) Church of St. Prokop in Bezděkov (1724 - 1727; Kilian Iznaz Dientzenhofer)
2) Church of St. Mary Magdalene in Božanov (1735 - 1743; Kilian Iznaz Dientzenhofer)
3) Church of St. Wenceslas in Broumov (1729; Kilian Iznaz Dientzenhofer)
4) The Chapel of Virgin Mary (1733; Kilian Iznaz Dientzenhofer)
5) Church of All Saints in Heřmánkovice (1722 - 1724; Kilian Iznaz Dientzenhofer)
6) Church of St. Barbara in Otovice (1725 - 1727; Christoph Dientzenhofer)
7) Church of St. Jakob the Greater in Ruprechtice (1720 - 1723; Christoph Dientzenhofer)
8) Church of St. Margaret in Šonov (1726 - 1729; Kilian Iznaz Dientzenhofer)
9) Church of St. Michael in Vernéřovice (1719 - 1722; Christoph Dientzenhofer)
10) Church of St. Anne in Vižňov (1719 - 1728; Kilian Iznaz Dientzenhofer)

The location of these churches is shown in Figure 7 below. However, in the scope of this work, only four churches included in Broumov group will be considered and their description is represented in the following paragraphs.

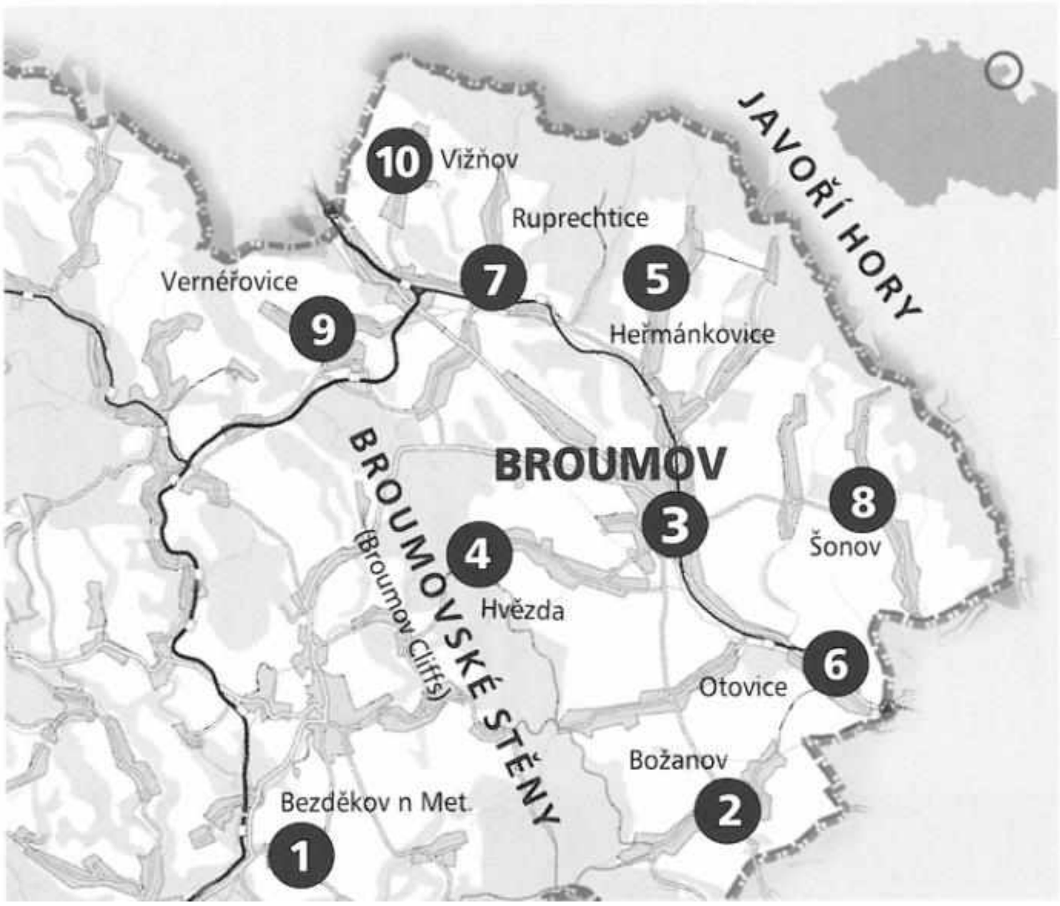


**Figure 7 – The location of Broumov group of churches (The Region Of Broumov, 2013)**

### 2.4.1 Church of St. Jacob the Greater in Ruprechtice

The Church of St. Jacob the Greater stands on the place of the original wooden small church, that was mentioned for the first time in 1386. In accordance with the preserved sources, the construction of a new church was commenced by laying down the foundation stone on August 18, 1720. The author of the project was probably Christoph Dientzenhofer, but the construction works were managed by his son Kilian Iznaz. In April 1723, the church was consecrated. The abbot, Otmar Zinke in this case, made decisions on construction of country churches within the monastic estate on his own, the costs, however, should have been borne at least partially by the local parish priests both from the income of the parish and from contributions of volunteers. For this reason, churches were rather simple and plain. When it comes to the church in Ruprechtice, it is a one bodied building with prolate octagonal floor projection divided by wide window bays. A new painting of St. Jacob the Greater was added to the church altar in 1998, the author of the painting being the painter and graphic artist Vjaceslav Iljašenko. The church stands on a hill in the northern part of the village in the middle of a churchyard, the renaissance two-storey bell tower built in the second half of the 16$^{th}$ century serving as a churchyard gate. Most original bells were embargoed during the First World

War. Next to the church, there is also a parsonage with the emblem of the Břevnov-Broumov abbey pictured above the entrance, (The Broumov Group of Churches, 2013).

**Figure 8 – The Church of St. Jacob the Greater before restoration (Region Broumovsko, n.d.)**

**Figure 9 - The Church of St. Jacob the Greater after restoration (Region Broumovsko, n.d.)**

### 2.4.2 St. Ann Church in Vižňov

The eldest source where the original wooden church in Vižňov was mentioned is dated by the first half of the 14th century. Before the Thirty Years' War, it was a parish church, then it became a filial one subject to Vernéřovice. The construction of the present of St. Ann church started in 1724 and was completely finished four years later. It was conducted by a master builder from Broumov, the architectural design was certainly provided by Kilian Iznaz Dientzenhofer. The body of the church has deep oval floor projection with middle sections of the side walls being slightly dished. Two similar parts of the building with round corners are attached to the shorter sides. On the west side, there is a small chancel, a low sanctuary attached to it. On the east side, there is an organ loft with two spiral staircases, a prismatic tower built next to it. The builder of the tower was Johann Heinrich Opitz from Ruprechtice. The hipped roof was originally covered with shingles. In 1903, it was replaced with slates and in the 1900s, with metal, red-painted profiles. Today, there is almost nothing left from the original equipment in the church because all valuable movables were deposited in the collection storage rooms after the baroque painting had been stolen from the church in 1990. The St. Ann Church in Vižňov, which should be considered as a very interesting building of high qualities, is unfortunately ruined to a high degree today, (The Broumov Group of Churches, 2013).

**Figure 10 - St. Ann Church in Vižňov (Region Broumovsko, n.d.)**

### 2.4.3 All Saints Church in Heřmánkovice

The Church stands on a small hill in the middle of a church-yard above the village and it is a dominant of the surrounding landscape. The foundation stone of the church was laid down on November 8, 1722, and abbot Otmar Zinke chose Kilian Iznaz Dientzenhofer to be the author of the project. The shell construction was finished and consecrated in 1723 and for another three years, they had been working on the completion of the building and the interior works. The floor projection of the church is one of oblong octagon with dished walls. The organ loft is followed by a tower with round corners, at the end of the body, there is a narrow chancel, behind which there is a rectangular sanctuary. The church ceiling is painted with a view to the heavens, portraits of the four evangelists are painted in semicircular fields and angels in elliptical bracing. The original painting was created shortly after the church erection; however, it was painted over in 1865. The inside equipment dates back to the period of baroque and empire, the more valuable movables however had to be moved out and stored safely in consideration of the growing number of robberies in the Broumov churches after 1990. In the close vicinity to Heřmánkovice church, the original baroque parsonage has been also preserved. The circumstances and the exact dates of its origin are not known, by the form and quality of its execution, however, the building of the parsonage is an excellent complement to the neighbouring building of the Dientzenhofer 's church, (The Broumov Group of Churches, 2013).

**Figure 11 - All Saints Church in Heřmánkovice (Region Broumovsko, n.d.)**

### 2.4.4 St. Barbara Church in Otovice

At the place of the Church of St. Barbara, there was originally a wooden chapel. It is obvious from the preserved building accounts that the construction of the new church in accordance with the design of Christoph Dientzenhofer started in 1725 and that it was completed within two years. In the years 1748-50, the works were still ongoing on the inside decoration and interiors. The church has a body of oval floor projection, into which seven semi-circular chapels were included counting the chancel. Also, the floor projection of the entrance hall is slightly conical. Above the main entrance to the church, a commemorative tablet is placed with the initials of abbot Otmar Zinke and the chronogram 1726 - i.e. the year when the Church of St. Barbara was consecrated. These days, the indoor installation of the church includes only the altar architecture. Probably the most valuable relic is the altar in the southern middle chapel, which used to be the great altar. Various embossing and sculptures dating back mostly to the 18$^{th}$ century and other more valuable movables are safely stored in collection storage rooms. Within the exposition of the Museum of the Broumov Region, the panel Crucifixion from the 15$^{th}$ century is exhibited, which is a unique relic of its kind in the whole region and originally, it was a part of the very church in Otovice. Today we can find the Church of St. Barbara in the middle of the village, but originally it was standing alone in the open space surrounded by the churchyard.

**Figure 12 –St. Barbara Church in Otovice (Region Broumovsko, n.d.)**

## 3. GEOLOGICAL CONDITIONS OF REGION

From the geological subsoil point of view, all churches are located in the area of the Paleozoic of the Bohemian Massif, which is mainly formed by rocks of the Permian age, in places of the Upper Carboniferous. These are mainly layers of sedimentary rocks, which are typically represented by red and grey mud (dusty claystones), sandstones, arkoses, conglomerates and in some places by coal seams. At the same time, there are also positions of igneous rocks in this area, which are represented by flowing rocks petrographically belonging to ignimbrites or paleo-rhyolites (quartz porphyries). All mentioned types of rock were found in individual drilled wells and were also confirmed by petrographic analysis using polarization microscopy, (Kuklík, Kovářová, & Záleský, 2021). The character of the geological subsoil for each church and its proximate surroundings are described in more detail in the following paragraphs. Geological map of Broumov region with the location of churches under examination is demonstrated in Figure 13.

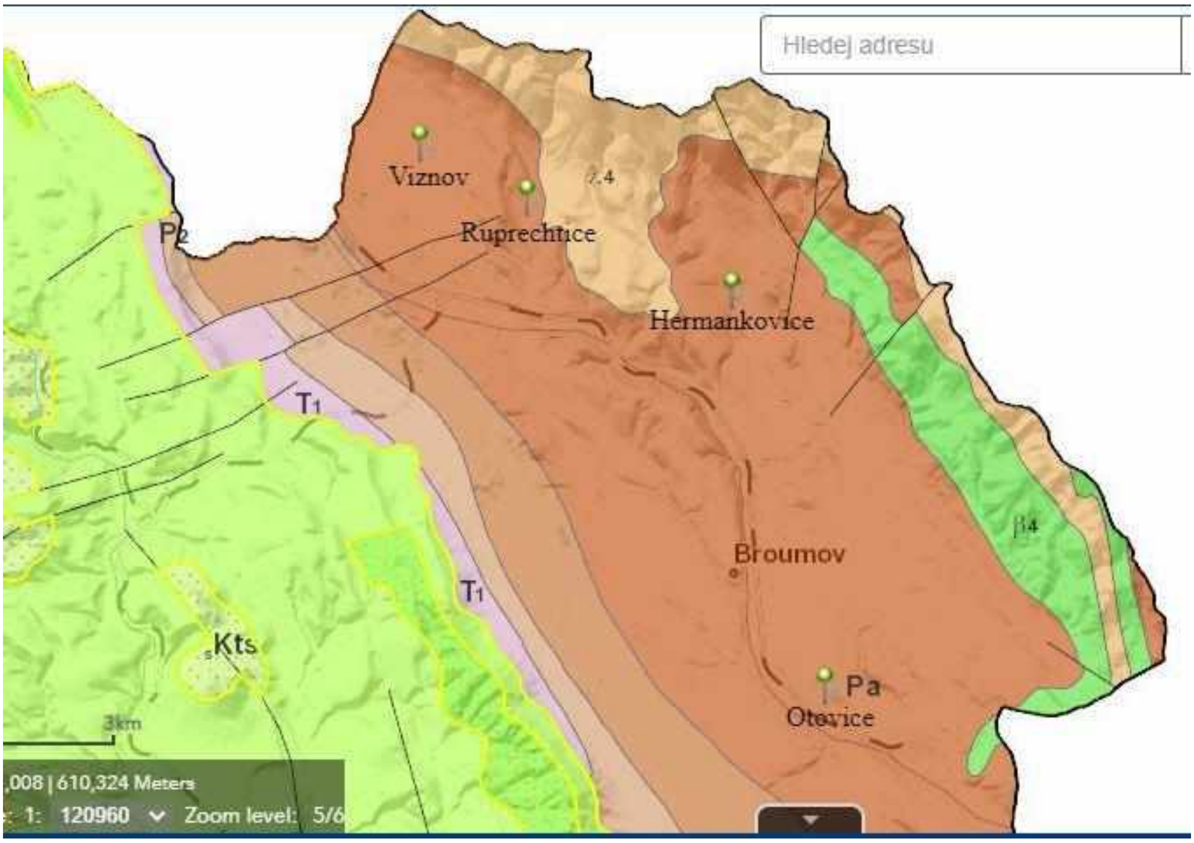


**Figure 13 – The geological map of Broumov region with location of churches (Ceska Geologicka Sluzba, n.d.)**

### 3.1.1 Church of St. Jacob the Greater in Ruprechtice

From the geological point of view, the St. Jakob the Greater Church in Ruprechtice is located in a place where unconsolidated fluvial sediments of Quaternary era occur. These are mainly sands and gravels. The area of Quaternary Aeolian unconsolidated sediments of Pleistocene age, which are loess clays, is very close to the locality of the church. The rocks, which are also located in the close proximity to the church, are part of the Young Paleozoic filling of the Inner-Sudeten basin presented by a volcanic rhyolite complex, for which transitions of rhyolite tuffs and ignimbrites are typical, (Kuklík, Kovářová, & Záleský, 2021).

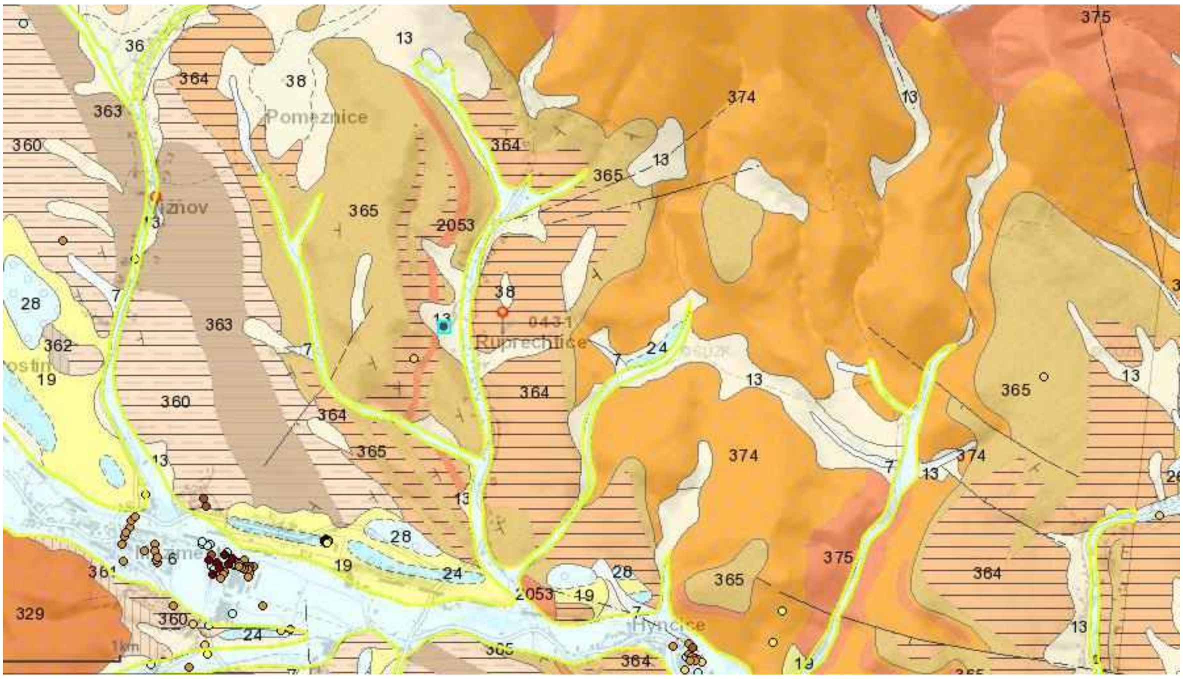


**Figure 14 -The geological map of Ruprechtice (Ceska Geologicka Sluzba, n.d.)**

The following types of soil are represented on the map: 24 - sand, gravel; 19 - loess clay; 28 - sand, gravel; 6 - alluvial sediment; 363 - variegated aleuropelites, often calcareous, with lenses and tubers of silicites, rarely with limestones; 360 - siltstones, clayey siltstones, dusty, sometimes fine-grained sandstones; 13 - stony to aluminous-stony sediment; 364 – aleuropelites, positions of arkoses and arkose-shaped sandstones, volcano-detritic breccias, sandstones and tuffs; 365 – volcano-detritic sandstones, breccias and conglomerates with positions of rhyolite tuffs and tuffites; 374 - rhyolite ignimbrites; 2053 - limestone, siltstones, siltstones, tuffites and andesite tuffs; 361 - limestones, brightly coloured aleuropelites, location of arkose to polymictic sandstones and conglomerates

### 3.1.2 St. Ann Church in Vižňov

From a regional-geological point of view, the locality is situated in the West Sudeten region of the Bohemian Massif, in the Inner Sudeten Basin. The rock base here consists of sediments and volcanic of the Martínkov, Olivětín and Noworud layers of the Broumov Formation (Lower Permian). There are rhyolite tuffs of ash streams, rhyolite ignimbrites and weakly sintered tuffs - transition between tuffs and ignimbrites (Noworud v.) in the north of Vižňov. To the east, rocks are volcanodetritic development - sandstones, arkoses, and offal. The Olivetín layers are characterized by reddish-brown and pastel-colored aleuropelites, siltstones and sandstones. Furthermore, the rocks of the Vižňov horizon (martínkovské v.) are represented by pastel siltstones and siltstones with layers of silicites and aleuropelites, inferiorly dusty and fine-grained sandstones. The cover formations here are diluvial sediments (slopes), proluvia, fluvial (stream) sediments, or their mutual combinations, (Kuklík, Kovářová, & Záleský, 2021).

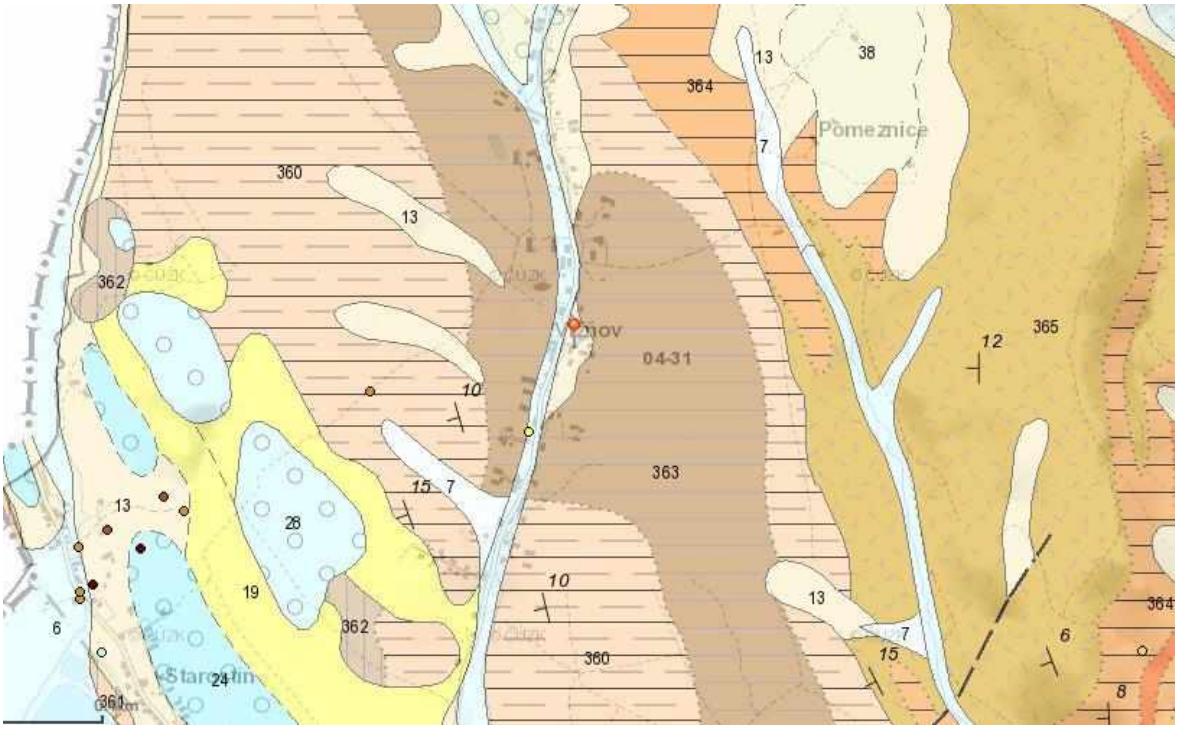


**Figure 15 - The geological map of Vižňov (Ceska Geologicka Sluzba, n.d.)**

The following soils are found in close proximity in correspondence with a map presented in Figure15: 6 - alluvial quaternary sediment; 363 - variegated aleuropelites, often calcareous, with lenses and tubers of silicites, rarely with limestones; 360 - siltstones, clayey siltstones, dusty, sometimes fine-grained sandstones; 13 - stony to aluminous-stony sediment of Quaternary era; diluvial and fluvial unconsolidated Quaternary sediment; 364 - aleuropelites, positions of arkoses and arkose-shaped sandstones, volcano-detritic breccias, sandstones and tuffs; 365 – volcano-detritic sandstones, breccias and conglomerates with positions of rhyolite tuffs and tuffites; 2053 - limestone, siltstones, siltstones, tuffites and andesite tuffs; 19 - loess clay (quaternary sediment); 362 - brightly colored aleuropelites with limestones and occasionally with silicites; 24 - sand, gravel (Quaternary age); 28 - sand, gravel (Quaternary age).

### 3.1.3 All Saints Church in Heřmánkovice

The location of All Saints Church geologically belongs to the Notch-East part the Inner-Sudeten basin. Orographically, the area belongs to the Broumov Highlands. Apart from the Quaternary sediments, all the rocks that rise to the surface belong to the Broumov Permian Formation. The formation is very colourful and facially variable, built by both igneous and sediments. The Broumov Formation, which exceeds a thickness of 800 m in the Broumov region, is divided from below into the Noworud, Olivětín and Martínkovice layers, while the geological subsoil of the church site is formed by the Olivětín strata. The average thickness of the Olivětín layers is about 200 m. The upper Olivetín strata is formed by two different lithic facies - the Walchia shale and the volcano-detritic facies, (Kuklík, Kovářová, & Záleský, 2021).

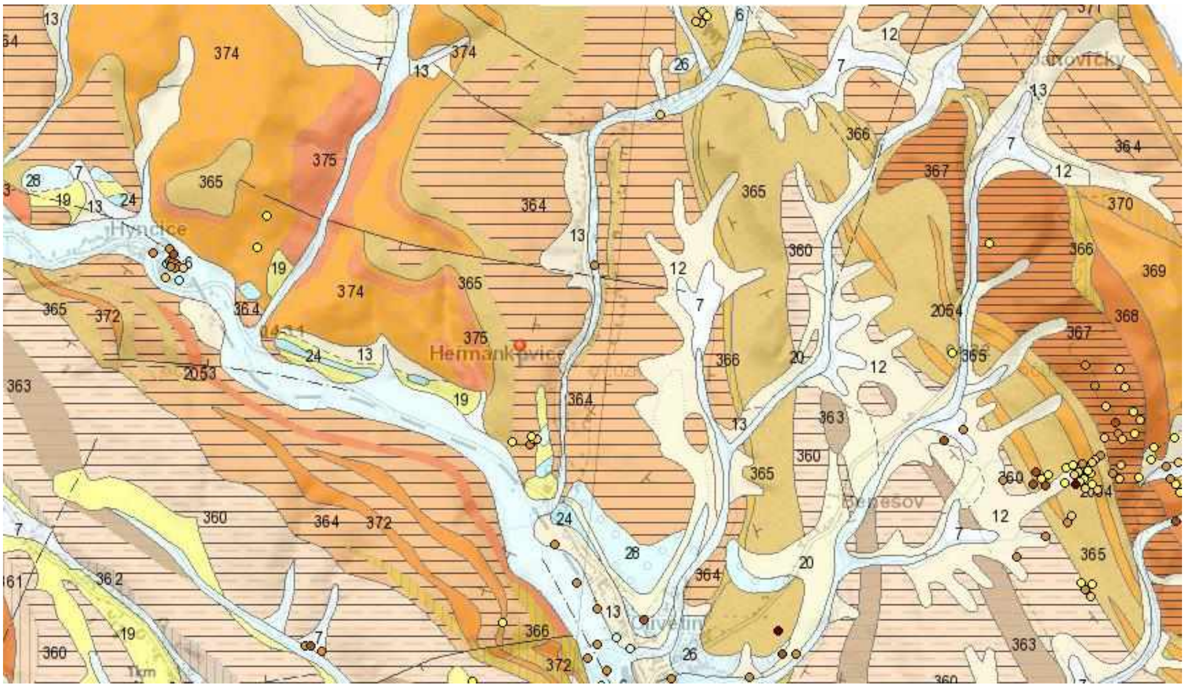

**Figure 16 - The geological map of Heřmánkovice (Ceska Geologicka Sluzba, n.d.)**

In the close proximity to the church, arkoses (364) and arkose sandstones, volcano-detritic breccias, sandstones and tuffs, volcano-detritic sandstones (365), breccias and conglomerates with layers of rhyolitic tuffs and rhyolitic ignimbrite (374) are located (Figure 16).

### 3.1.4 St. Barbara Church in Otovice

The bedrock of St. Barbara Church is formed mainly by rocks of Paleozoic era, which are mostly presented by sediments of Permian era, like so-called Broumov siltstones, sandstones and claystones. These rocks were found relatively close to the surface and directly under the foundations of the church itself. The bedrock is mainly formed by weathered silty claystones to claystones. Their colour changes from red brown to grey with depth. Each layer is characterized by a different state of compaction and degradation of the rock material due to the weathering action. The surface layer is represented by sandy soil with small fragments of claystone and other sedimentary rocks in this area, (Kuklík, Kovářová, & Záleský, 2021). The geological situation is shown in the following Figure 17.

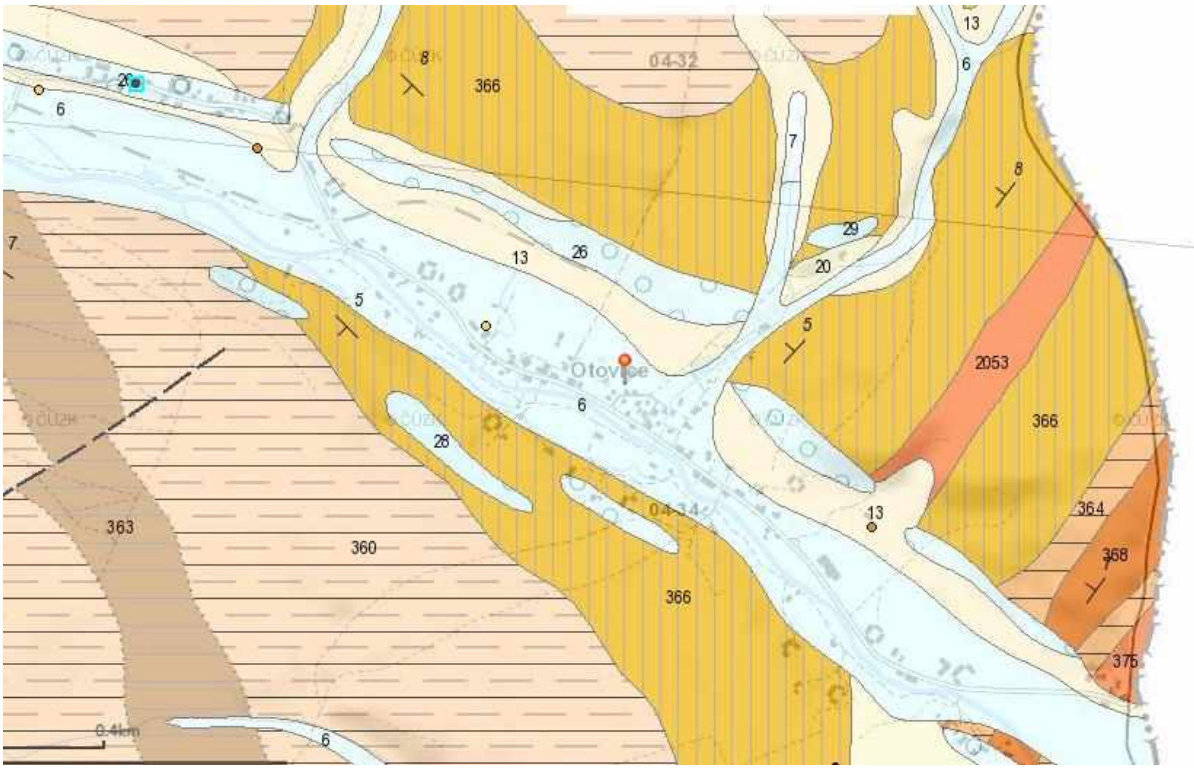


**Figure 17 - The geological map of Otovice (Ceska Geologicka Sluzba, n.d.)**

The stones in the closest surrounding are 336 - grey to greenish grey silty claystones , in some places bituminous limestones, siltstones and fine-grained sandstones;360 - siltstones and clay siltstones, silty fine-grained sandstones; 368 – andesitic tuffs and agglomerates; 2053 – limestones, marlites, siltstones, andesitic tuffs; 364 – aleuropelites, arkose and arkose sandstones positions, volcano-detritic breccias, sandstones and tuffs; 375 – rhyolitic tuffs and rhyolites

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# 4. SOIL-STRUCTURE INTERACTION

In structural engineering, the modelling of superstructures is often detailed while the foundation and sub-soil structure is simplified and assigned as pinned or rigid supports. This assumption is applicable in stiff soils where in comparison the upper structure is relatively light. Nevertheless, as applied to soft soils with heavy buildings, this effect should be considered. It refers to historical structures with quite thick walls and shallow foundations.

In order to model soil-structure interaction (SSI) the following approaches are usually used:

1) Structural approach - modelling a structure as beam/plate element laying on elastic subsoil layer
   a) Winkler model
   b) Pasternak model
2) Continuum approach

Both approaches have advantages and disadvantages. The first method is easy to implement in practice and it is less time consuming while the second one is mainly used when more realistic solution is required. However, the estimation of material properties for representation of subgrade is a well-known problem. In contrast to the structural approach, the soil parameters are straight forward to specify for an elastic continuum model but implementing such models in existing commercial software is problematic. Nevertheless, both categories require geotechnical evaluation of soil parameters, (Aron & Jonas, 2012).

The detailed description of all methods is represented below:

- **Winkler Method**

The Winkler model is simple and is used for many engineering problems. This idealized model represents the subsoil as independent springs on a rigid base neglecting the effect of shear deformation between the springs, as shown in Figure 18, (Skar, Klar, & Levenberg, 2019).
Mathematically, it can be expressed as:

$$p = w \cdot k \tag{1}$$

Where p is the reaction force or contact pressure, w is a displacemnt and k is known as a coefficient of subgrade reaction and the only parameter that is required to describe the charcteristics of the given soil. That is why the model is also called “One-parameter model”. The value of k depends not only on elastic properties of subsoil but also on dimentions of the area acted upon by the subgrade reaction. The concept of subgrade reaction was intoduced into applied mechanics by Winkler in 1857, (Terzaghi, 1955).

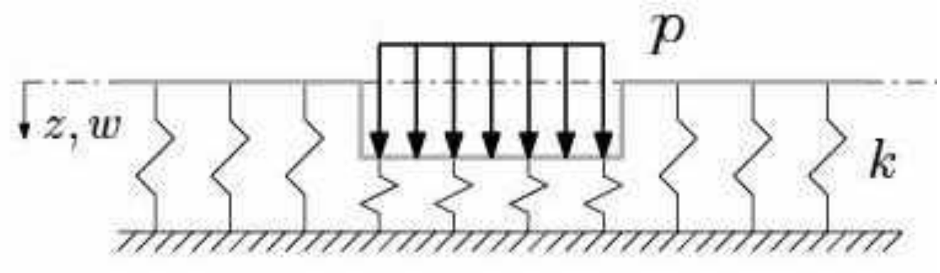


**Figure 18 -Winkler Method. Adapted from (Skar, Klar, & Levenberg, 2019)**

As it has been already mentioned, the primary deficiency of the model is that the shear capacity of the soil is negleted. As a result, the displacement has no spread in transverse direction. Therefore, the displacement discontinuity appears between loaded and unloaded surfaces However in reality, soil has shear capacity and no displacement discontinuity occures as shown in Figure 19, (Aron & Jonas, 2012).

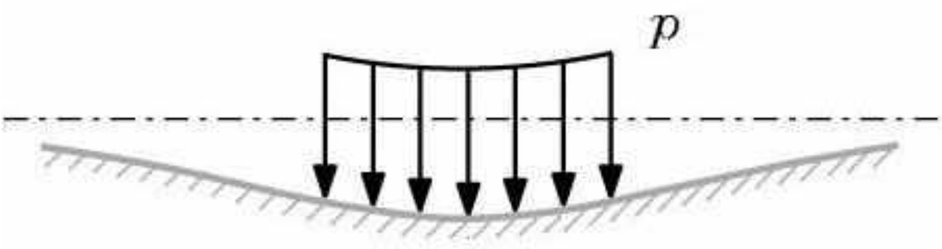


**Figure 19 – Displacement observed in reality. Adapted from (Aron & Jonas, 2012)**

- **Pasternak Model**

In order to overcome some of the drawbacks of the Winkler model, different solutions were proposed to incorporate the effect of shear, widely known as the "Two-Parameter Models".
However, Winkler Pasternak that assumes the end of the springs to be connected by a beam or plate undergoing only transverse shear deformation remains the most widely used model. The continuity provided by this model provides arises from the shear component introduced, (Breeveld, 2013).

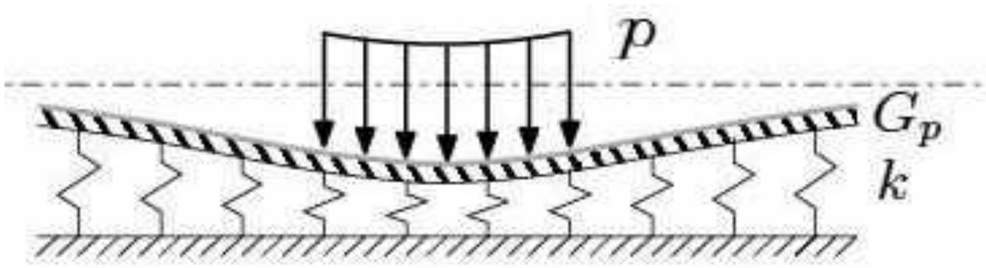


**Figure 20 - Pasternak model. Adapted from (Skar, Klar, & Levenberg, 2019)**

Commonly used expression for Pasternak model is shown below

$$p = w \cdot k - G_p \frac{d^2 w}{dx^2} \tag{2}$$

where p is a contact pressure. W represents a displacement in the vertical direction, k is a modulus of subgrade reaction and Gp is a shear modulus of a shear layer. Physically, this parameter represents the interaction due to the shear action among the spring element. The value of Gp is related to the shear modulus G, but they are not the same. The Gp value can be found using the following equation

$$G_p = G \cdot H \tag{3}$$

where H- is effective depth over which the soil undergoes shear (Breeveld, 2013)
Another equation commonly used for Pasternak model is shown below

$$f_z = C_1 w - C_2 \Delta w \tag{4}$$

where the left side of equation represents the vertical load applied on the layer. W is a displacement in the vertical direction, C1 and C2 are constants which represent compressive and shear deformability.

- **Continuum Approach**

In continuum mechanics, continuum is defined by a continuously distributed matter through the space. The simplest elastic continuum is described with the constitutive relation with linear elastic isotropic behaviour given by Hooke's law. Without failure criteria the elastic medium has infinite tension and compression capacity, which can be questioned for soil. Several constitutive relations exist, with different failure criteria in tension and compression, which better capture soil behaviour, e.g. Mohr-Coulomb's constitutive relation (Aron & Jonas, 2012).

There are two methods mainly used to model the continuum: Finite Element Method (FEM) and Boundary Element Method (BEM). FEM is preferable for non-linear soil properties, the method is well-known and several commertial softwares are based on it. In linear elastic analysis BEM is suitable to consider infinite and semi-infinite spaces, (Aron & Jonas, 2012).

Several analytical solutions exist for continuum approach. One of them is commonly used for obtaining stress distribution in elastic media and It is called Boussinesq solution meanwhile the other one is known as Westergaard solution.

Boussinesq gave a solution for the stress distribution in an elastic medium subjected to concentrated load on surface. The following assumptions are applied for this method:

- The soil mass is an elastic continuum, having a constant value of modulus of elasticity E, i.e. ratio between the stress and strain is constant.
- The soil is homogeneous and isotropic, i.e. it has the same properties at different points and in all directions.
- The soil mass is semi-infinite, i.e. it extends to infinity in the downward and lateral directions
- The soil is weightless and free from residual stresses before the load application.

The vertical stress at a point P at the depth z from the point load Q at the for Boussinesq solution is shown below

$$\sigma_z = \frac{3Q}{2\pi z^2} \frac{1}{\left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}} = \frac{Q}{z^2} I_b \tag{5}$$

The coefficient $I_b$ is known as the Boussinesq influence coefficient for the vertical stress. It can be defined from the given value of r/z, (Arora, 2004).

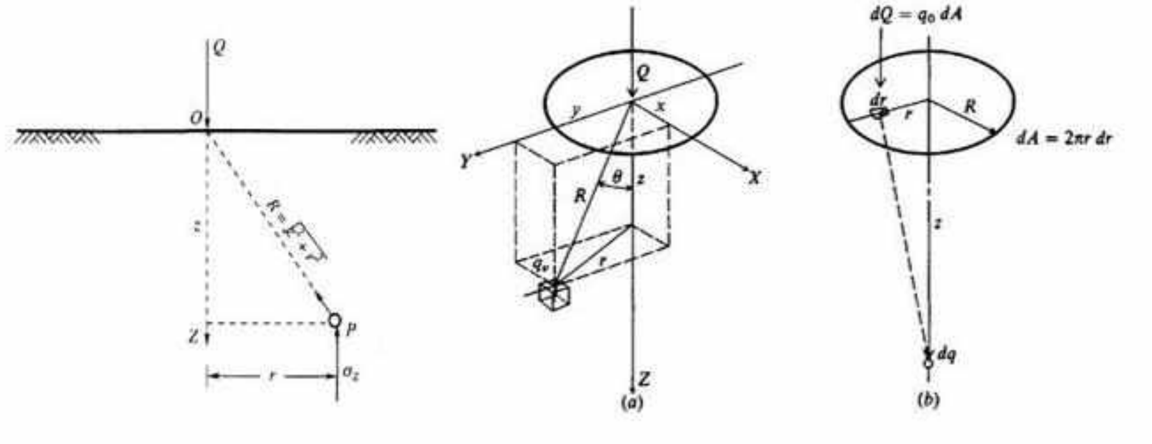

**Figure 21 - Intensity of pressure based on Boussinesq approach (Bowles, 1997)**

Boussinesq's solution assumes the soil deposit is isotropic. Actual sedimentary deposits are generally anisotropic. There are generally thin layers of sand embedded in a homogeneous stratum. It is taken into account in Westergaard's solution. The following assumptions are applied:

- There are thin sheets of rigid materials sandwiched in a homogeneous soil mass. These thin layers are closely spaced and of infinite rigidity and they are therefore incompressible.
- The soil medium is anisotropic
- Only downward displacement of soil mass is possible and a whole without any lateral displacement
- It considers Poisson's ratio in a range between 0.0 and 0.50 for the elastic material

According to Westergaard, the vertical stress at a point P at the depth z below the concentrated load Q is given by formula below

$$\sigma_z = \frac{Q}{z^2}\frac{1}{\pi\left[1+2\left(\frac{r}{z}\right)^2\right]^{3/2}} = \frac{Q}{z^2}I_w \tag{6}$$

Where $I_w$ is know as a Westergaard influence coefficent. The value of $I_w$ is significantly smaller than the Boussinesq influence factor $I_b$, (Arora, 2004).

## 4.1 Elastic layer theory

The purpose of the analytical solution is to determine the deformation of the elastic layer in the vertical direction. The solution procedure relies on neglecting horizontal displacements similar to the standard assumptions applied to the analysis of Westergard subspace. Certainly, this assumption results in a stiffer soil response thereby providing an upper estimation of the depth of influence zone, (Kuklík, 2010). The problem formulation is shown in Figure 22.

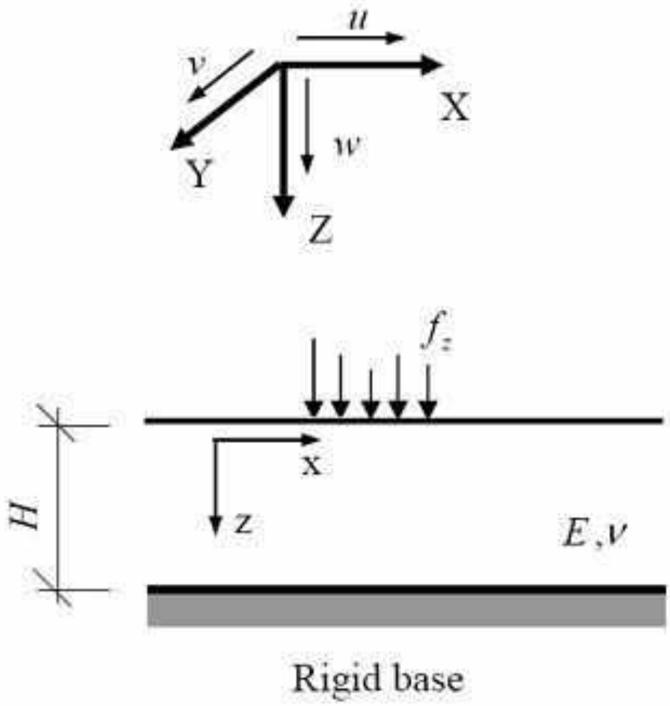


**Figure 22 – The formulation of elastic layer solution (Kuklik & Brouček, 2011)**

## 4.2 Depth of influence zone

It is known that the effect of the load applied on the soil extends only to a certain depth that is called an influence zone. Its determination is an important task in geotechnical engineering.
An influence zone is an area or rather a domain below the acting load. In this domain all the deformations due to the applied load take place. Below the influence zone all the deformations from the particular load are negligible. The depth of influence zone depends on the size of the load, its magnitude, and the characteristics of soil, (Brouček, 2013).

Due to excavation to a certain depth h, the original geostatic stress state, which sets the initial compaction of soil represented by the preconsolidation pressure, the highest stress level in the soil recorded during the prior loading history, is reduced. Subsequent surcharge at the footing bottom gives further redistribution of vertical stress. It is assumed that in the region where the vertical effective stress due to surcharge at the footing bottom combined with the reduced geostatic effective stress (by excavation) does not exceed the original geostatic effective stress, the skeleton deformations are negligible. This condition describes the depth of the influence zone H (Figure 23), (Kuklik & Brouček, 2011).

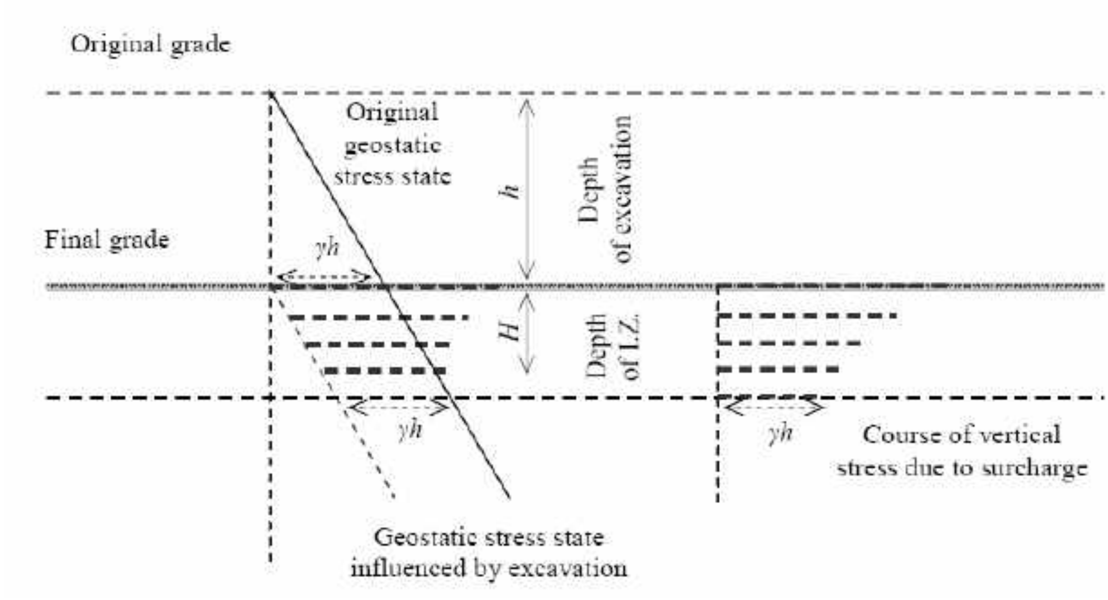


**Figure 23 – The governing idea of influence zone depth calculation (Kuklík, 2010)**

The estimation of depth of the influence zone is based on the deformation of an elastic layer. However, it should be noted that firstly, the influence zone is a proportional to the strip load with width of 2a. Secondly, the influence zone does not depend on Young’s modulus, but there is a significant role of Poisson’s ratio, (Kuklik, 2010).

$$H = \frac{\pi a}{2}\sqrt{\frac{2-2\upsilon}{1-2\upsilon}}\frac{1}{\ln\left(sin\frac{\pi\gamma h}{2f_z}+1\right)-\ln\left(\cos\frac{\pi\gamma h}{2f_z}\right)} \tag{7}$$

$f_z$ is the value of surface load, $\gamma h$ represents the value of pre-consolidation.

The settlement W(mm) was calculated using the expression for the rigid infinite strip footing laying on the flexible layer.

$$w_o = \sum_{n=0}^{\infty} \frac{fz}{2(2n+1)\sqrt{C_1 C_2} + (2n+1)^2 2aC_1}. \qquad (8)$$

Where C1 and C2 are found using the formulas 9 and 10 below

$$C_1 = \frac{\sqrt{H}.\pi^2}{8H^2} E_{oed} \quad (9) \qquad\qquad C_2 = \frac{\sqrt{H}}{2} G \quad (10)$$

$C_2$ parameter as mentioned above describes the shear resistance of the subsoil and it should be less or equal to parameter $C_1$.

The theory implemented in the GEO 5 software is very similar to the above mentioned and it is called the Theory of Structural strength. The structural strength represents the resistance of soil against deformation for a load at the onset of failure of its internal structure. With decreasing of m coefficient, the soil respond tends to be linear.

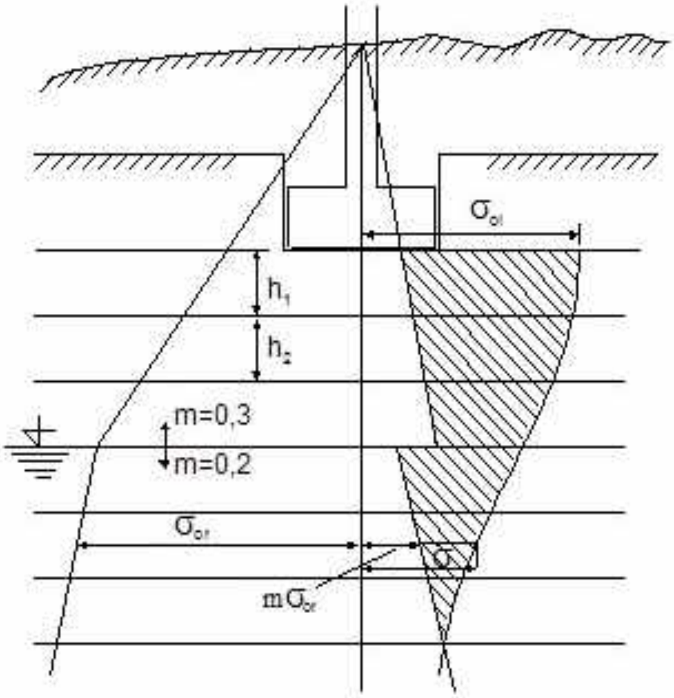


**Figure 24 – The depth of influence zone based on the theory of structural strength (GEO5, n.d.)**

The influence zone is defined as a depth below the foundation bottom at which the increment of vertical stress $\sigma_z$ is equal to the structural strength of soil.

$$\sigma_z = m \cdot \sigma_{or} \tag{11}$$

where m is a coefficient of structural strength and $\sigma_{or}$ is an original geostatic stress.
The settlement S is determined by dependence below

$$s = f(\sigma_z\ , m\ , \sigma_{or}) \tag{12}$$

# 5. 2D MODELLING IN GEO5

FEM GEO 5 software was used tasked to perform a numerical analysis. GEO FEM is geotechnical software based on the finite element method. The software is able to simulate the excavation as well as stages of construction by activating and deactivating the elements. Moreover, it proposes a great number of soil model to simulate non-linear, time dependent behaviour.
GEO5 allows to model various types of problems and analyses under the assumption of plain strain deformation.

The analysis was carried out in four steps for the purpose of simulation of construction process. The first stage represents the original condition of soil, second stage takes into account excavation of soil, third stage shows the construction of foundation and the last step is the application of load from the upper structure. The mesh was generated with size of element equal 0.3 m.

Due to existed restrictions which led to inability of inspection of the churches and lack of time allotted for the project, the load for every church was assumed based on the load summary represented in master theses from previous years. All loads from the superstructure were directly applied to the foundation without modelling of the walls for this reason, it does not consider the staging of wall construction. Since the GEO 5 program allows to import models, they were all developed in AutoCAD to speed up the process.

To represent the soil beneath the church, a Modified Mohr-Coulomb material model was used. Additionally, a Modified Linear elastic model was applied for the church footings. The material properties of masonry foundation used for Linear elastic model are presented in Table 1.

**Table 1 – Material properties of foundation**

| Parameter | Value |
|---|---|
| E (MPa) | 2000 |
| Poisson’s ratio | 0.2 |
| ρ (kN/m$^3$) | 20 |

## 5.1 Material models

Generally, the material models can be divided into two categories: linear and non-linear. Two basic linear material models adopted in GEO5 software – Elastic and Modified Elastic. A linear model is the basic material model that assumes a linear relationship between the stress and strain given by the Hooke law. In a one-dimensional problem the Hooke law describes the linear dependence of stress $\sigma$ on strain $\varepsilon$ via the Young modulus E (Figure 25). In this framework the linear model provides a linear variation of displacements as a function of applied loads, (GEO5, n.d.).In order to use this model, the following data is required: $\gamma$ – unit weight of soil, v – Poisson's ratio, E- modulus of deformation.

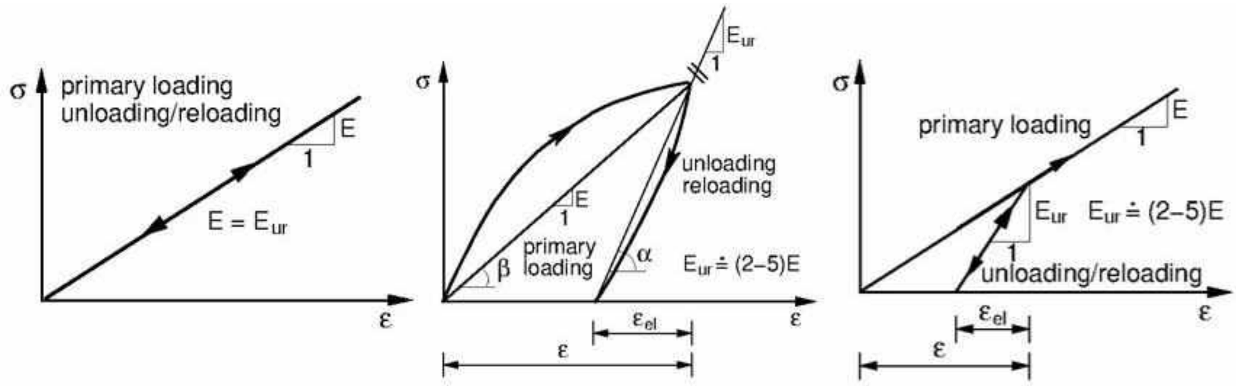


**Figure 25 – Stress-strain diagrams for: Linear elastic model (left); Realistic soil behaviour (middle); Modified elastic model (right) (Sejnoha, 2009)**

However linear behaviour of soil is only acceptable for relatively low magnitudes of applied load. The Modified linear model attempts at least to some extent to take this into account by considering different modulus for loading and uploading. Therefore, the Modified Elastic is characterized by $E_{def}$ or $E_{sec}$ and $E_{ur}$. A drop in the material stiffness along the given loading path attributed to the plastic yielding is reflected through a modulus of elasticity E, which can be imagined as a secant modulus associated with a certain stress level. An elastic response is assumed upon unloading. To increase the clarity of model formulation the elastic modulus for the unloading branch is replaced by the unloading-reloading modulus $E_{ur}$ that governs the response of a soil upon unloading and subsequent reloading up to the level of stress found in the material point prior to the unloading, (GEO5, n.d.).
The value of unloading-reloading Young's modulus $E_{ur}$ is approximately three times of $E_{def}$. The typical stress-strain diagrams are shown in Figure 25.

The second category of material models represents a real soil behaviour better. It can be divided into Mohr-Coulomb failure criterion and critical state of soil. The Mohr-Coulomb Failure criterion is well-known and widely used in the geotechnical engineering. This criterion includes the following models: Drucker-Prager, Mohr-Coulomb and Modified Mohr Coulomb which are capable of modelling hardening and softening behaviour. A common feature of these models is the evolution of unbounded elastic strains when loaded along the geostatic axis (GEO5, n.d.). The concept of critical state of soil is represented by Modified Cam-clay, Generalized Cam clay and Hypoplastic models.

Employing non-linear models allows to capture the typical nonlinear response of soils. These models describe the evolution of permanent (plastic) deformation of a soil material. The onset of plastic deformation is controlled by so-called yield surface. The yield surface can be either constant (elastic-perfectly plastic material), or it can depend on the current state of stress (material with hardening/softening). Unlike the modified linear model, the nonlinear models require specifying only the elastic modulus. A drop in the material stiffness is a result of evolution of plastic strains and corresponding redistribution of stresses. This consequently yields an instantaneous tangent material stiffness as a function of the current state of stress represented in the Figure below by an instantaneous tangent modulus $E_T$, (GEO5, n.d.).

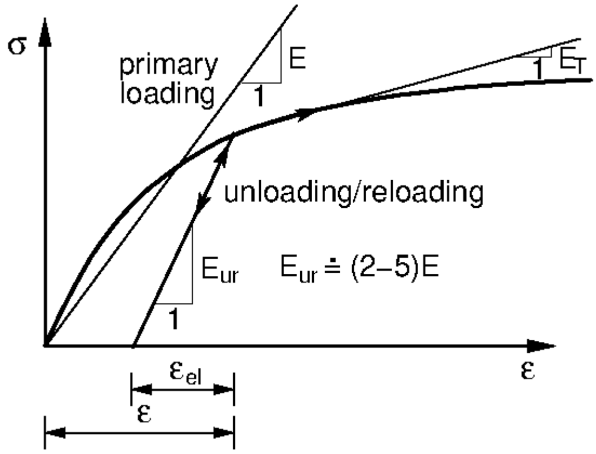


**Figure 26 – Stress-strain diagram for non-linear models (GEO5, n.d.)**

Additionally, to the basic material mentioned above, non-linear models require the following characteristics: φ - angle of internal friction (in degrees), c – cohesion (in kPa) and ψ - dilatation angle(degrees). The first two define the onset of plastic deformation, while the third parameter controls the development of plastic volumetric strain(dilatation) in the soil.

## 5.2 Soil data

In this section, geological conditions and soil properties used for modelling and further calculations are presented.

Two boreholes of the depth up to 12 meters were drilled for each church to build the geological profiles. In order to simplify the modelling process, all layers were grouped in bigger macro groups according to their similarity. The original borehole profiles are presented in the Annex A.

Due to absence of laboratory test results, the physical and mechanical properties of soil for this work were assumed based on visual inspection and the recommendations provided by the Department of Geotechnics of CTU in Prague.

The simplified geological sections and properties of soil assumed are demonstrated for every church particularly.

### 5.2.1 Church of St. Jacob the Greater in Ruprechtice

The church is situated on the plain surface without any significant changes in level. Figure 27 demonstrates the layout of boreholes drilled for St. Jacob church. First borehole is located on the left side of back part of the church whereas the second one was drilled on the facade part.

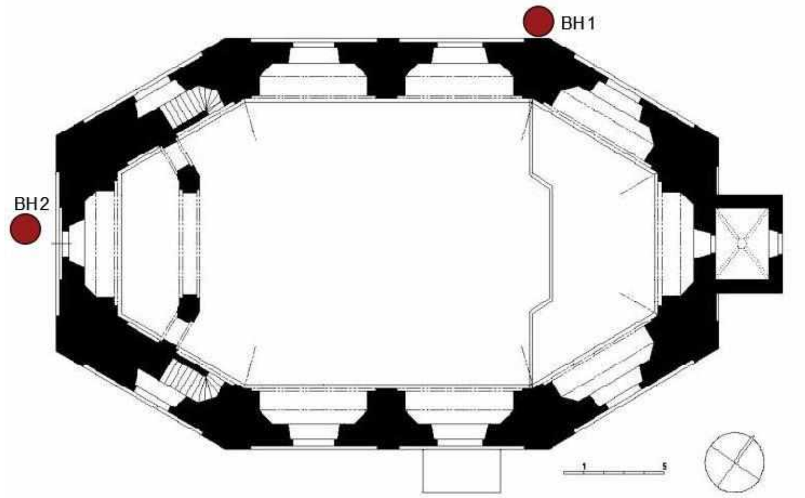


**Figure 27 - The layout of boreholes**

The original profiles were divided into three soil groups and the geological section built on this simplification is presented in Figure 28 while suggested soil properties are shown in the table below.

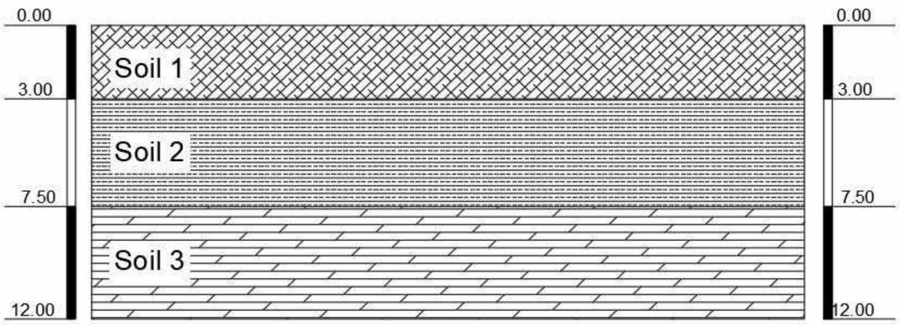


**Figure 28 - The geological section of St. Jacob The Greater Church**

**Table 2 – Physical and mechanical properties of soil for the Church of St. Jacob the Greater**

| Soil type | E def, (MPa) | Poisson's ratio | Density, (kN/m3) | Angle of friction, φ, (°) | Cohesion, c, (kPa) |
|---|---|---|---|---|---|
| **Soil 1** | 20 | 0.35 | 19.5 | 25 | 30 |
| **Soil 2** | 25 | 0.35 | 20.0 | 30 | 30 |
| **Soil 3** | 30 | 0.35 | 20.5 | 30 | 40 |

### 5.2.2 St. Ann Church in Vižňov

Figure 29 presents the location of boreholes based on which further, the geological profiles were obtained. One borehole was drilled on the right side of church at the back of the corner while the second one was situated on the left from the facade of church.

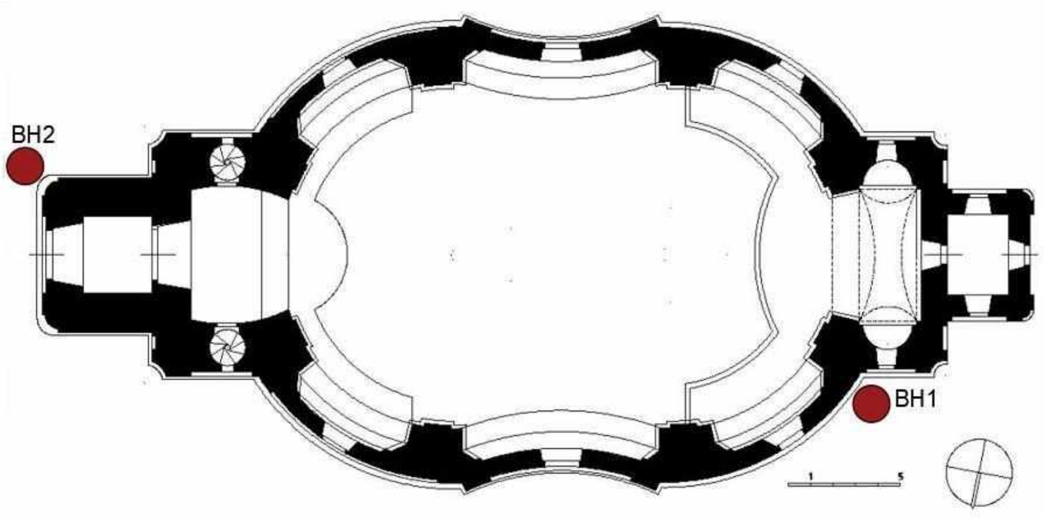


**Figure 29 - The location of boreholes**

The developed geological section is shown in Figure 30 and Table 3 contains physical and mechanical properties of soil.

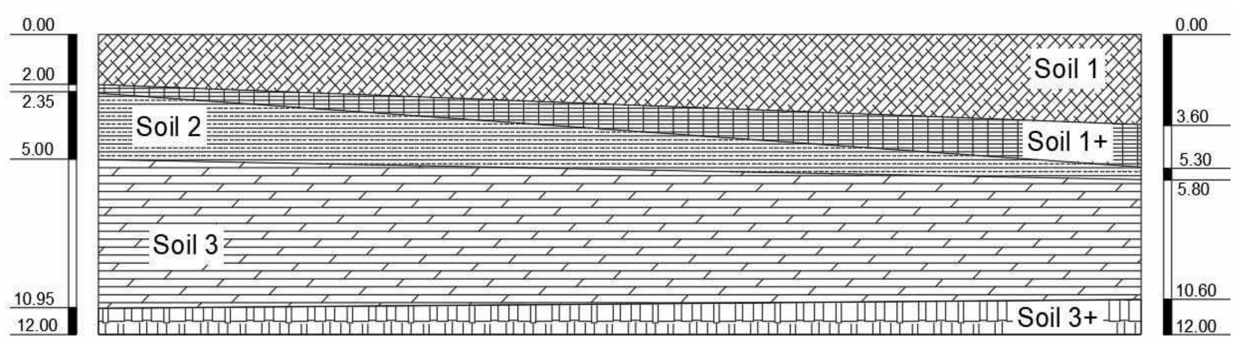


**Figure 30 - The geological section of St. Ann Church**

**Table 3 – Physical and mechanical properties of soil for St. Ann Church**

| Soil type | E def, (MPa) | Poisson's ratio | Density, (kN/m3) | Angle of friction, φ, (°) | Cohesion, c, (kPa) |
|---|---|---|---|---|---|
| **Soil 1** | 20 | 0.35 | 19.5 | 25 | 30 |
| **Soil 1+** | 25 | 0.35 | 19.5 | 25 | 30 |
| **Soil 2** | 25 | 0.35 | 20.0 | 30 | 30 |
| **Soil 3** | 30 | 0.35 | 20.5 | 30 | 40 |
| **Soil 3+** | 35 | 0.35 | 20.5 | 30 | 50 |

### 5.2.3 All Saints Church in Heřmánkovice

Two boreholes were drilled for the All Saints Church, one of them from the south part of church and another one is on the north side (Figure 31). The developed profile is shown in Figure 32

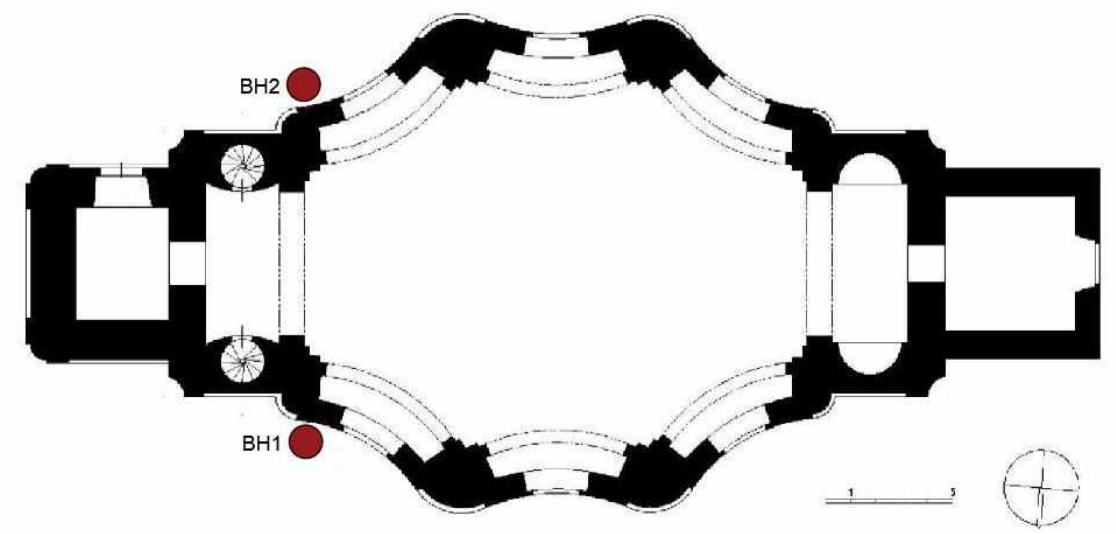


**Figure 31 - The layout of boreholes**

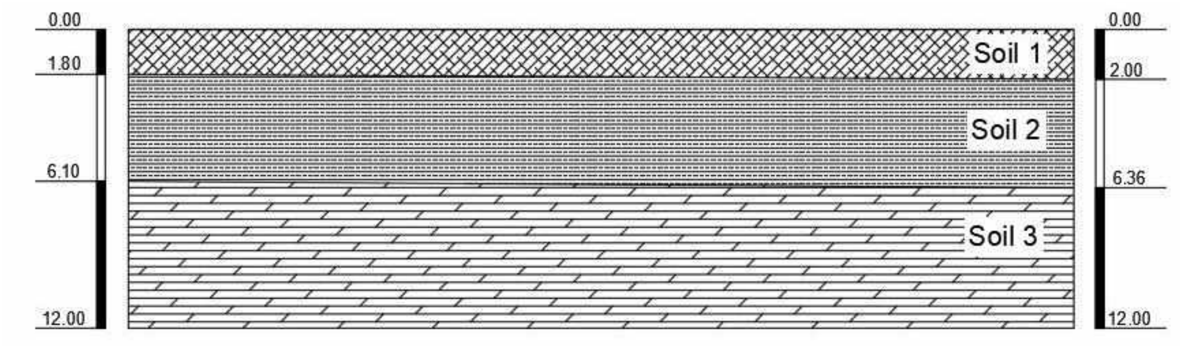


**Figure 32 – The geological profile of All Saints Church**

Physical and mechanical properties of soil for the All Saints Church summarized in the table below

**Table 4 – Physical and mechanical properties of soil for All Saints Church**

| Soil type | E def, (MPa) | Poisson's ratio | Density, (kN/m3) | Angle of friction, φ, (°) | Cohesion, c, (kPa) |
|---|---|---|---|---|---|
| **Soil 1** | 20 | 0.35 | 19.5 | 25 | 30 |
| **Soil 2** | 25 | 0.35 | 20 | 30 | 30 |
| **Soil 3** | 50 | 0.35 | 22 | 40 | 60 |

### 5.2.4 St. Barbara Church in Otovice

St. Barbara Church is located on almost horizontal surface. Figure 33 demonstrates the location of boreholes (one to the left of church facade and another one on the right side of back part of the church) drilled during the investigations while Figure 34 shows the geological section obtained after simplification. Assumed properties of soil are presented in the Table 5 below.

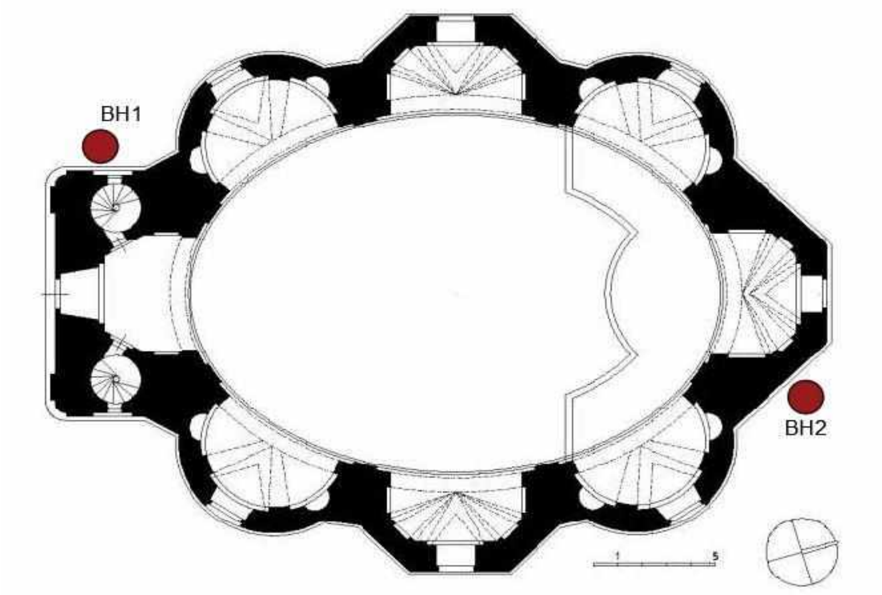


**Figure 33 - Location of boreholes**

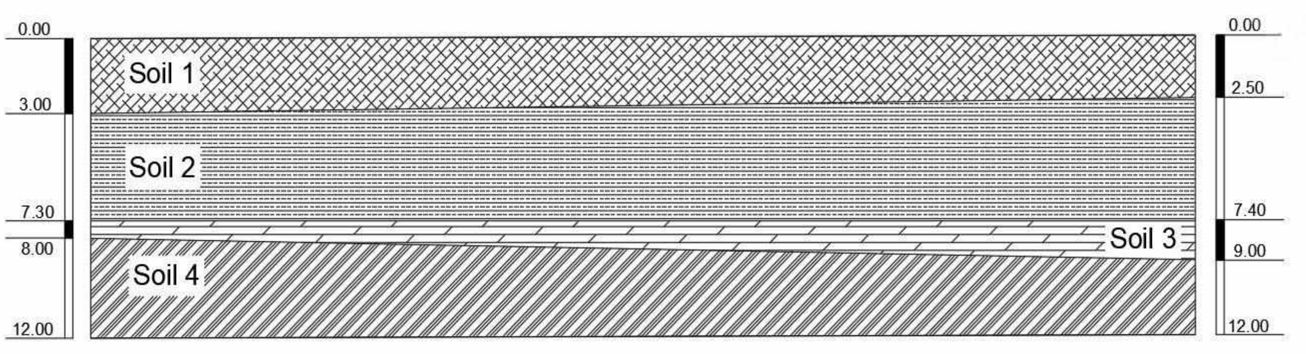


**Figure 34 - The geological section of St. Barbara Church**

**Table 5 - Physical and mechanical properties of soil for St. Barbara Church**

| Soil type | E def, (MPa) | Poisson's ratio | Density, (kN/m3) | Angle of friction, φ, (°) | Cohesion, c, (kPa) |
|---|---|---|---|---|---|
| **Soil 1** | 20 | 0.35 | 19.5 | 25 | 30 |
| **Soil 2** | 25 | 0.35 | 20.0 | 30 | 30 |
| **Soil 3** | 30 | 0.35 | 20.5 | 30 | 40 |
| **Soil 4** | 30 | 0.35 | 20.5 | 30 | 50 |

## 5.3 Calculation process

The analytical approach was compiled following steps: homogenization of modulus of elasticity, calculation of influence zone depth, estimation of vertical displacement.

For the purpose of calculation of the depth of influence zone the DEPTH software developed by prof. Kuklík was used. The program has a simple interface and requires the minimum initial data to calculate the depth of influence zone (Figure 35). It based on the theories described in Paragraph 4.2 of this work.

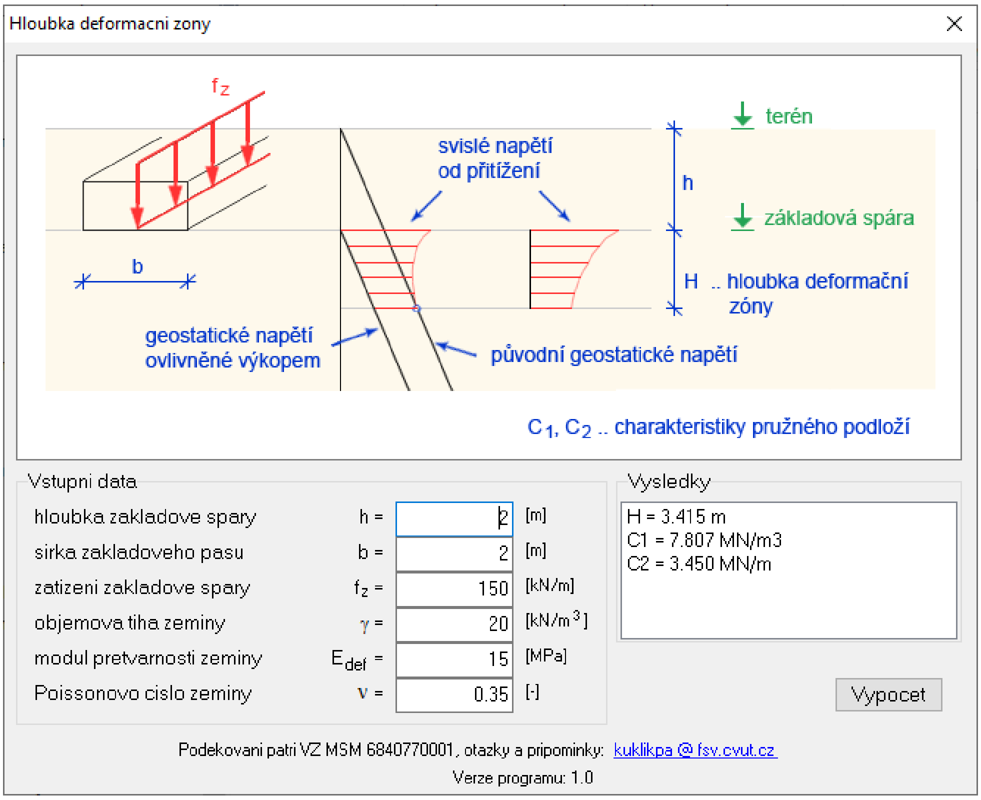


**Figure 35 – The interface of depth software**

Due to the program is able to perform calculations only for one soil layer, the homogenization of subsoil was carried out.

The calculation process includes few iterations. Firstly, the depth of influence is assumed as borehole depth and based on this data, the value of equivalent modulus was calculated. Next using this modulus and other soil parameters as well as depth and width of foundation, it is possible to compute the necessary value of depth of influence zone. Afterwards, the values of equivalent modulus should be recalculated.

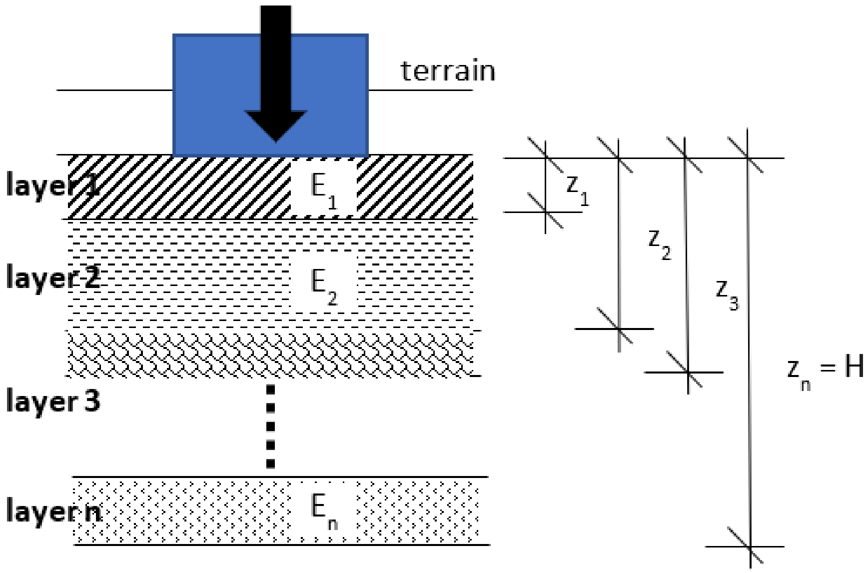


**Figure 36 – Scheme used for homogenization of subsoil**

Besides, in order to take into consideration, the effect of pre-consolidation the depth of foundation was increased by two times for St. Barbara Church and by three times for other churches.

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# 6. RESULTS

### 6.1.1 Church of St. Jacob the Greater in Ruprechtice

The church has a prolonged octagon plan with dimensions of 35 x 21 meters. The height of walls is equal to 15 meters while the thickness of the wall is 1.2 meters. The assumed load is based on calculations performed by Jacopo Scacco during his master thesis in 2018 and equals to 440 kN/m. The depth of foundation is equal to 1.5 meters.

Modulus of deformation received after homogenization equals 20.79 MPa. This value was utilized for further evaluation in DEPTH software. In order to take into consideration, the effect of pre-consolidation, the depth of foundation was increased threefold.

Table 7 shows the depth of influence zone and subgrade coefficients based on which the vertical displacement was determined. Calculations were performed for several load cases to see the difference in results.

**Table 6 - Calculation results for $E_{hom}$ =20.79 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 4.485 | 9.585 | 4.267 | 16.47 |
| 440 | 4.961 | 8.898 | 4.460 | 18.90 |
| 500 | 5.670 | 8.080 | 4.722 | 22.68 |

According to the table, the depth of influence zone received through the DEPTH software is equal to 4.96 meters while the corresponding vertical displacement is 18.90 mm.

Using the previous results, the graph below was built. It shows the load-displacement diagram which has a slight curvilinear character by that representing non-linear behaviour of soil.

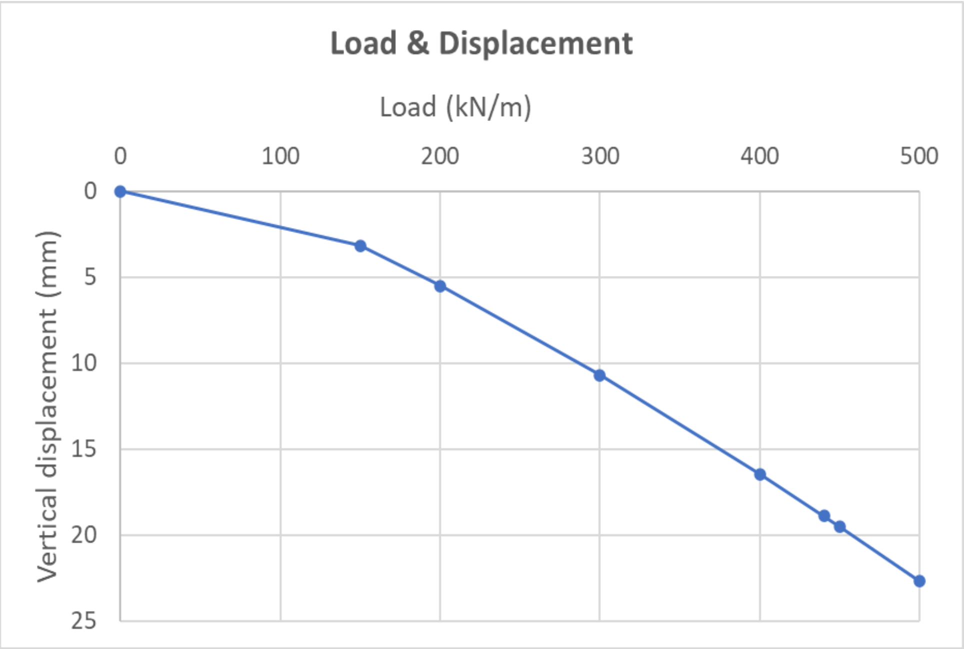


**Figure 37 - Load - Displacement diagram**

Figure 38 shows the 2D model developed for GEO5 software with dimensions of 37 meters long and 12 meters wide. Stress diagrams are presented in Figures 39-40 on the other hand the vertical displacement is demonstrated in Figure 41.

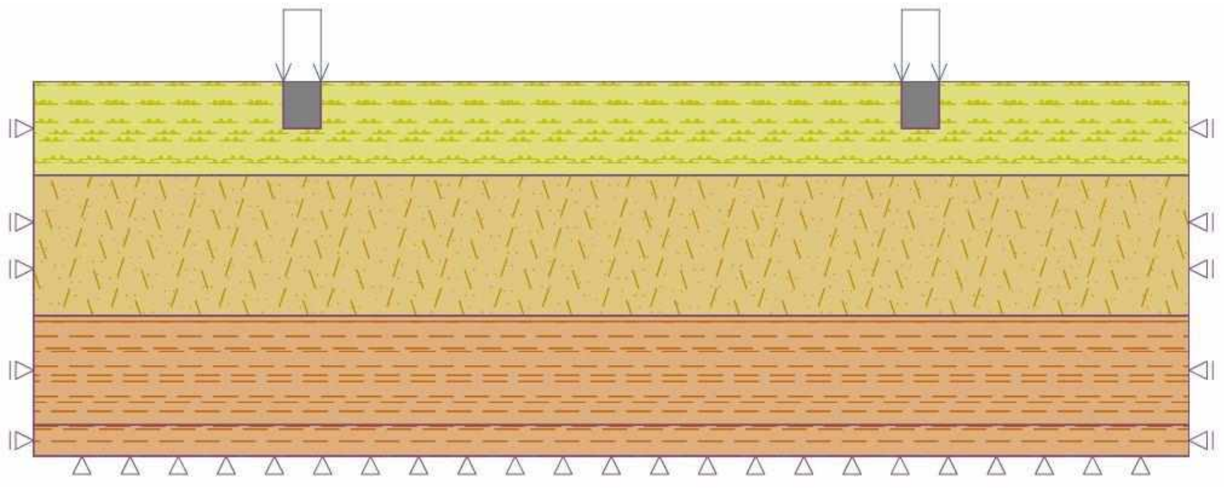

**Figure 38 - 2D model**

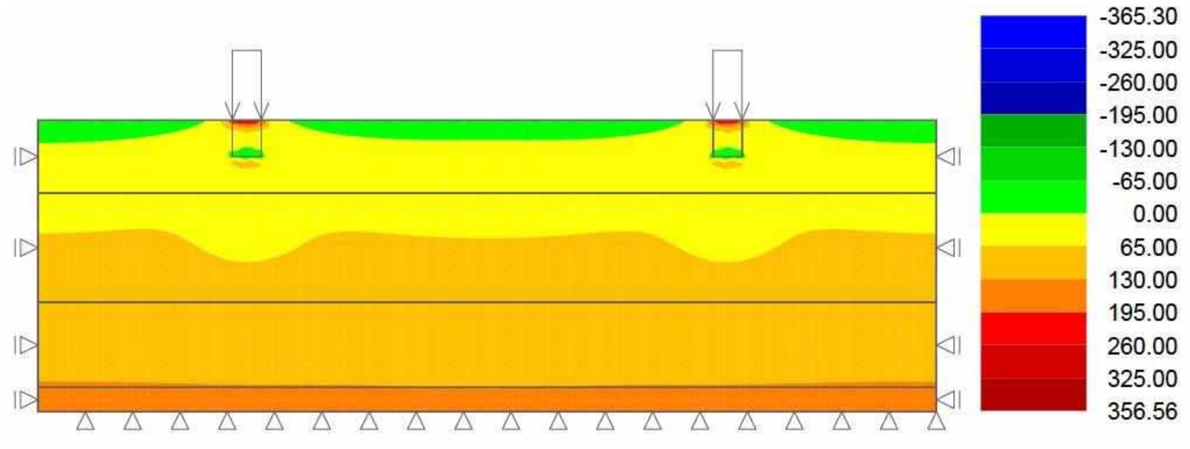


**Figure 39 – Total stress ϭx, kPa**

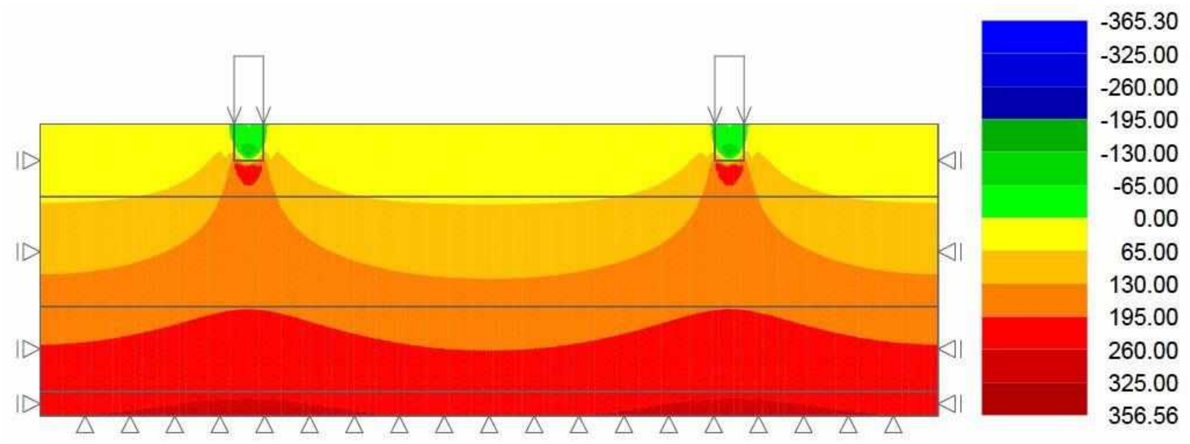


**Figure 40 – Total stress ϭz, kPa**

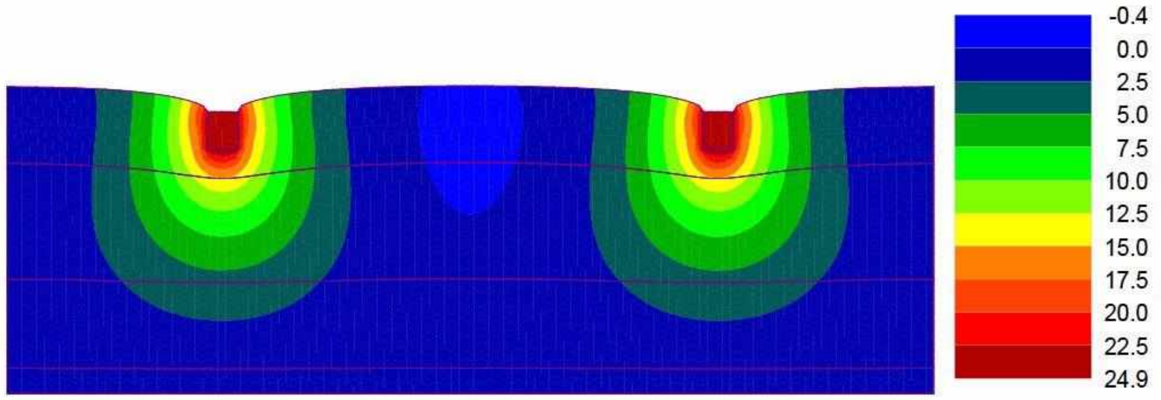


**Figure 41 - Vertical displacement, mm**

The maximum vertical displacement found in GEO5 is equal to 24.9 mm while the depth of influence zone is approximately 5.9 meters.

The following Figure 42 shows the evolution of plastic deformation. Stress concentration occurs under the edges of rigid foundations. Large vertical stresses lead to the formation of the ultimate stress state of the soil under the edges within small zones where plastic deformations are developed.

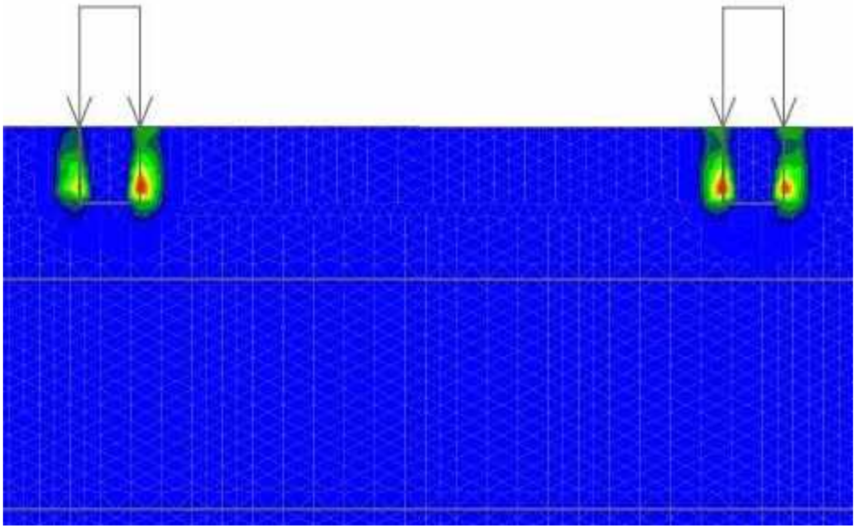

**Figure 42 - Plastic deformation**

### 6.1.2 St. Ann Church in Vižňov

The church is 20 meters wide and 43 meters long, the walls thickness is 1.2 meters. The height of tower is equal to 28 meters whereas the height of the walls is about 16 meters. The foundation depth is found at the level of 1.8 meters. The applied load was assumed as 460 kN/m for church and 730 kN/m for the tower part. Their values were taken in accordance with the work carried out by Pratik Gajjar during his master thesis in 2018.

The value of influence zone depth was calculated for both boreholes due to its deference in the thickness of the layers. Keeping in mind, that the effect of pre-consolidations should be considered, the depth of foundation was enlarged up to three times All calculations were performed for various loads. The results are presented for both loads (Tower and main part).

- **Borehole 1 (BH1)**

The homogenized modulus of elasticity for the BH1 and load equals 460 kN/m was found as 22.91 MPa so all further calculations were defined for this value.

**Table 7 - Calculation results for $E_{hom}$ =22.91 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.985 | 12.226 | 4.298 | 13.71 |
| 460 | 4.286 | 10.928 | 4.607 | 16.85 |
| 600 | 5.670 | 8.904 | 5.204 | 24.69 |
| 730 | 6.943 | 7.727 | 5.662 | 32.44 |
| 800 | 7.626 | 7.248 | 5.886 | 36.76 |

In accordance with the results, the following values were found for depth of influence and for the settlement – 4.286 m and 16.85 mm respectively.

Regarding to the load equals 730 kN/m, the homogenized modulus of elasticity for the BH1 is 21.42 MPa and outcomes based on this value are presented in Table 8.

**Table 8 - Calculation results for $E_{hom}$ =21.42 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.985 | 11.468 | 4.019 | 14.63 |
| 460 | 4.286 | 10.217 | 4.308 | 18.02 |
| 600 | 5.670 | 8.325 | 4.865 | 26.41 |
| 730 | 6.943 | 6.312 | 7.225 | 34.63 |
| 800 | 7.626 | 6.777 | 5.503 | 39.32 |

The vertical displacement is 34.63 mm and the corresponding value of depth of the influence zone is defined as 6.943 m.

- **Borehole 2 (BH2)**

The same estimations were performed for borehole 2. Modulus of elasticity obtained after homogenization for the BH2 and load cade -460 kN/m is equal to 21.30 MPa that is why this value was used in the following evaluations. The estimation results are presented in Table 9.

**Table 9 - Calculation results for $E_{hom}$ =21.30 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.685 | 10.794 | 3.782 | 15.55 |
| 460 | 4.286 | 9.616 | 4.054 | 19.15 |
| 600 | 5.67 | 7.835 | 4.579 | 28.06 |
| 730 | 6.943 | 6.8 | 4.983 | 36.87 |
| 800 | 7.626 | 6.378 | 5.179 | 41.78 |

Based on the table, the influence zone depth and the vertical displacement for the main part load are equal to 4.286 m and 19.15 mm respectively

In Table 10 below, the results obtained for load equals 730 kN/m are summarized. As it can be seen, the influence zone depth for the load of 730 kN/m is equal to 6.943 and the vertical displacement is 34.89.

**Table 10 - Calculation results for $E_{hom}$ =20.16 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.685 | 11.404 | 3.996 | 14.71 |
| 460 | 4.286 | 10.16 | 4.284 | 18.12 |
| 600 | 5.67 | 8.278 | 4.838 | 26.56 |
| 730 | 6.943 | 7.184 | 5.265 | 34.89 |
| 800 | 7.626 | 6.739 | 5.472 | 39.54 |

It is evident that the difference in vertical displacement is very small due to proximity in equivalent modulus of elasticity. At the same time, the depth of influence zone remains identical, because it does not depend on the Young's modulus.

Load – Displacement diagram was developed for all four equivalent modules of elasticity and represented below.

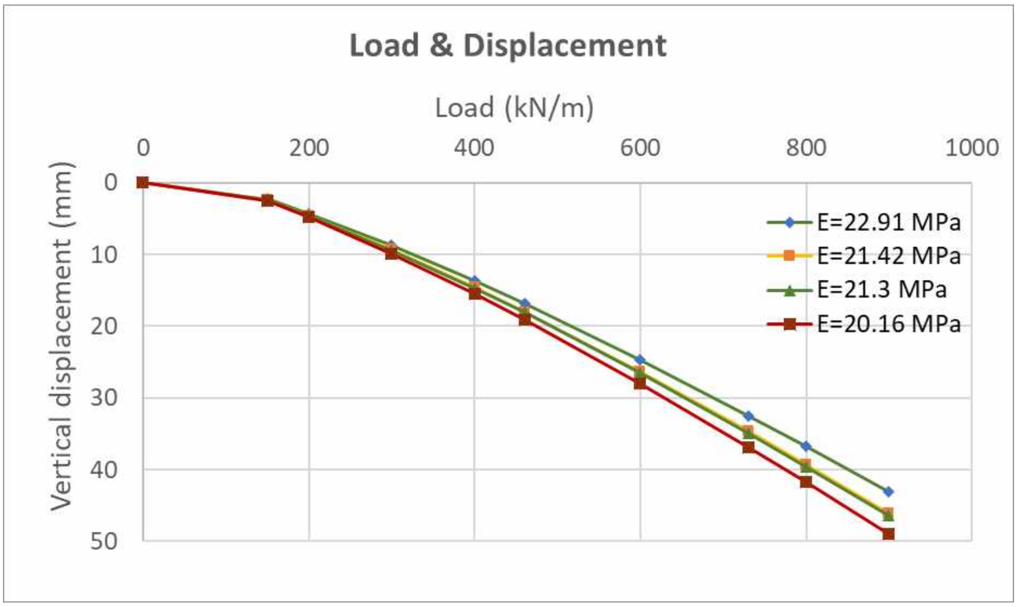


**Figure 43 – Load- Displacement diagram**

The same as for previous church, the diagram has a modestly curved character which expresses the non-linear behaviour.

Afterwards, the numerical calculations by means of GEO 5 software were carried out in order to compare results. The Figure 44 shows 2D model with dimensions of 36 x 12 meters used for calculation in FEM software. In this case two models were rum. The first model has the same uniform load for both footings which represents the main part of church while the second one has one foundation loaded with a greater load from tower part. However, the results are presented only for the second model since it takes into consideration both loads. The rest of diagrams could be found in the Annex B.

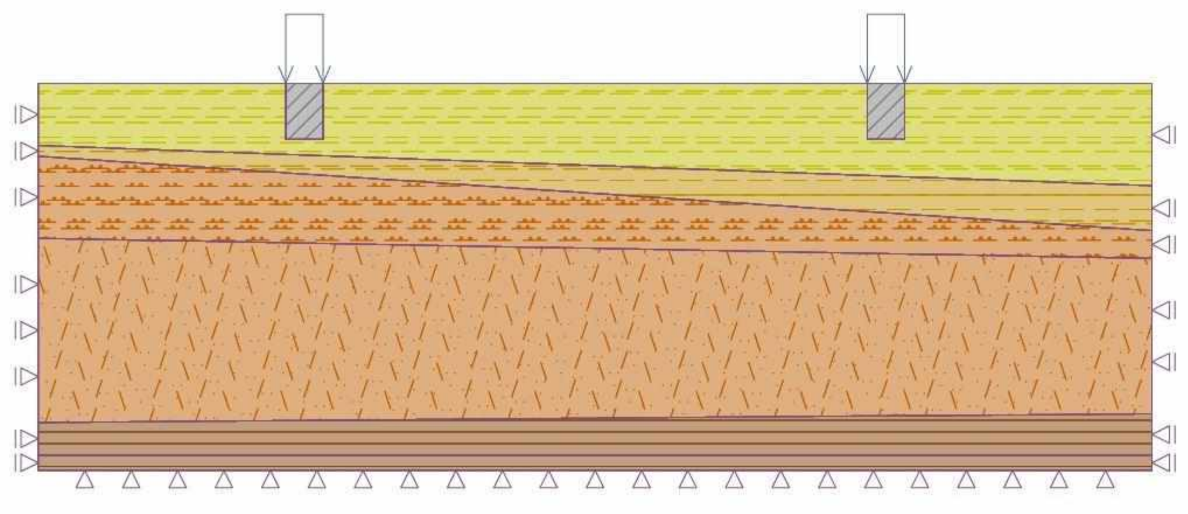

**Figure 44 - 2D model**

The obtained vertical displacement is presented in Figure 47 while stress diagrams are shown in Figures 45-46.

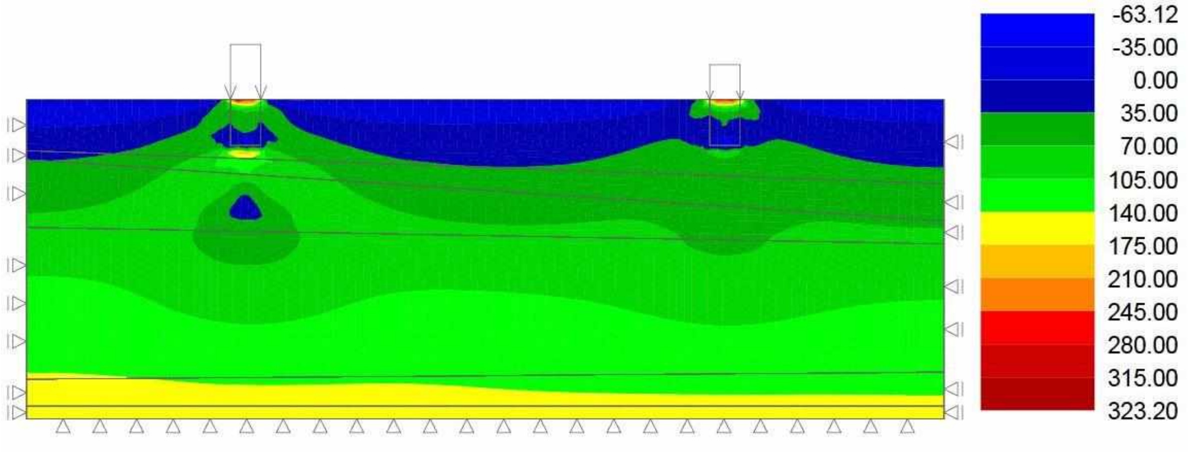


**Figure 45 - Total stress σx, kPa**

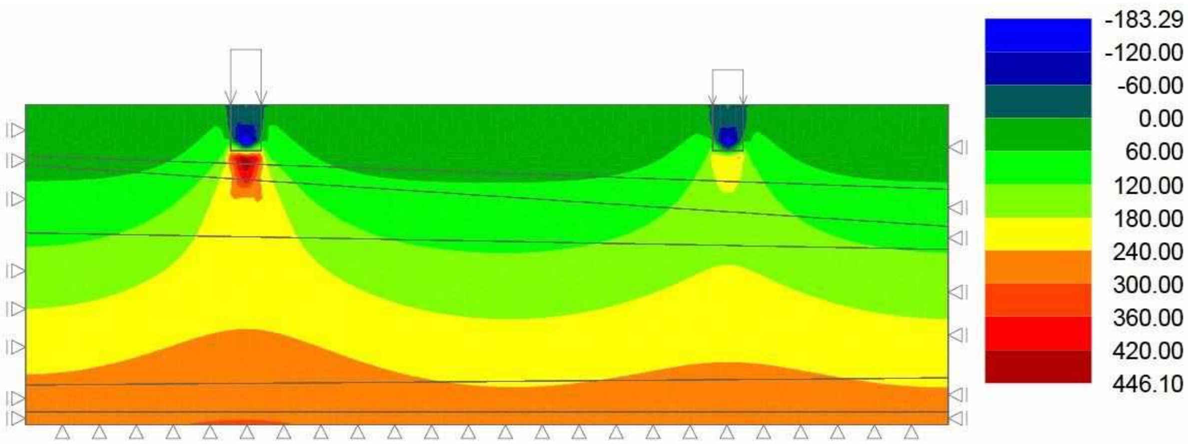


**Figure 46 - Total stress σz, kPa**

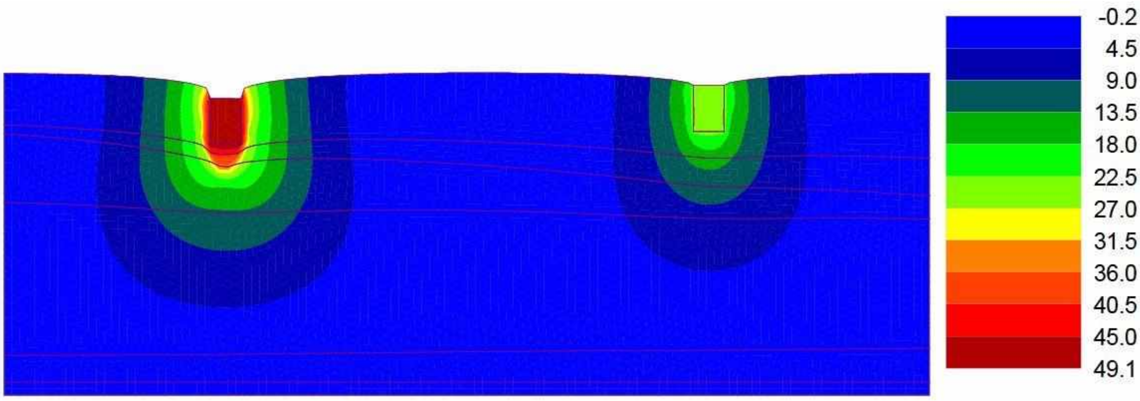


**Figure 47 - Vertical displacement, mm**

The maximum vertical displacement for the tower part was found 49.1 mm when for the church wall it is 24.5 mm. Regarding the depth of the influence zone, it is about 7.56 m and 5.85 m for the load of 730 kN/m and 460 kN/m receptively.

The development of zone of plastic deformation is shown for both loads in Figure 48.

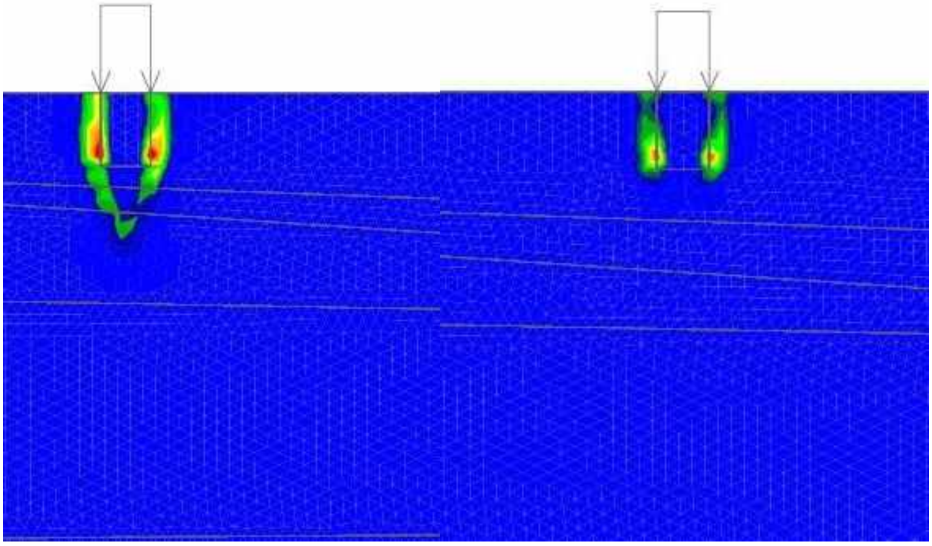

**Figure 48 – Plastic deformation: for load of 730 kN/m(left); for load of 460 kN/m (right)**

As it was mentioned before, the development of plastic zone is associated with the stress concentration. As the load increases, the plastic zones also increase as it can be seen in Figure 48. However, their growth is impeded by the horizontal ground resistance placed on the sides of them which is compacted within the stress zone. This resistance is gradually raised to a certain value and, called passive earth pressure. The increase in horizontal soil resistance is followed by a slowdown in the growth of those plastic deformation zones although it is accompanied by ever-greater deformations of the soil on the sides of most stressed zone of foundation. When the plastic deformation zone merges at a certain depth under the foundation, the compacted core is formed (Figure 48, left).

### 6.1.3 All Saints Church in Heřmánkovice

The church has a prolonged octagon plan with dimensions of 44 x 20 meters. The height of walls is equal to 15 meters while the thickness of the walls is 1.2 meters with the exception of sacristy wall where the thickness is equal to 1.0 meters. The depth of footing is found as 1.65 m below the ground. The load taken in calculation is based on the calculation performed by Evi Susanti in her dissertation work dated 2017 and it equals to 490 kN/m for the main part and 700 kN/m for the tower.

The Young's modulus obtained after homogenization is 21.87 MPa for the smaller load (main part) and 23.12 MPa for the larger load (tower part). In order to consider the pre-consolidation pressure, the foundation depth was multiplied by three. The results of both calculations are tabulated below (Tables 11-12).

**Table 11 - Calculation results for $E_{hom}$ =21.87 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.943 | 11.113 | 4.235 | 14.78 |
| 490 | 4.897 | 9.45 | 4.665 | 19.90 |
| 600 | 6.049 | 8.119 | 5.104 | 26.53 |
| 700 | 7.09 | 7.271 | 5.452 | 32.84 |
| 750 | 7.609 | 6.93 | 5.613 | 36.08 |

The obtained vertical displacement for the load case 490 kN/m is 19.90 mm meanwhile the relevant depth of influence zone is about 4.90 meters.

**Table 12 - Calculation results for $E_{hom}$ =23.12 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.943 | 11.748 | 4.477 | 13.99 |
| 490 | 4.897 | 9.99 | 4.931 | 18.83 |
| 600 | 6.049 | 8.583 | 5.396 | 25.09 |
| 700 | 7.09 | 7.687 | 5.764 | 31.06 |
| 750 | 7.609 | 4.326 | 5.934 | 48.94 |

For the greater load, the values of settlement and depth of influence zone are following – 31.06 mm and 7.09 meters. The load-displacement diagram is shown in Figure 49. It has the same slightly curved form as previously.

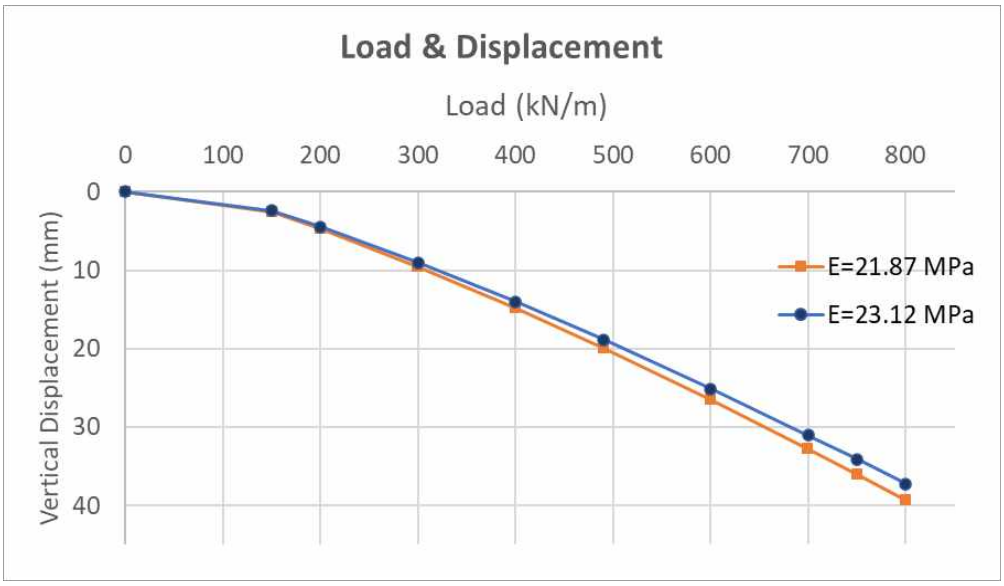


**Figure 49 – Load-Displacement diagram**

The Figure 50 shows the 2D model developed for GEO5 software. The model size was set with dimensions 37x12 meters. With regard to the different load, two models were run. As for St. Ann Church, one model has foundations with application of equal load while the second model has one foundation loaded with higher loading. In the text of report, the results are presented only for the second model, still the other diagrams are added to Annex B.

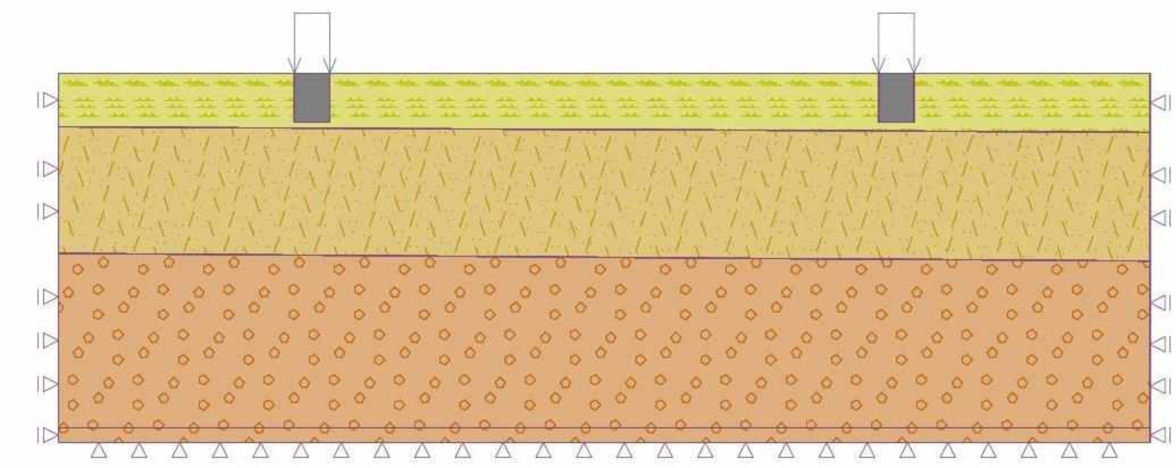

**Figure 50 - 2D model**

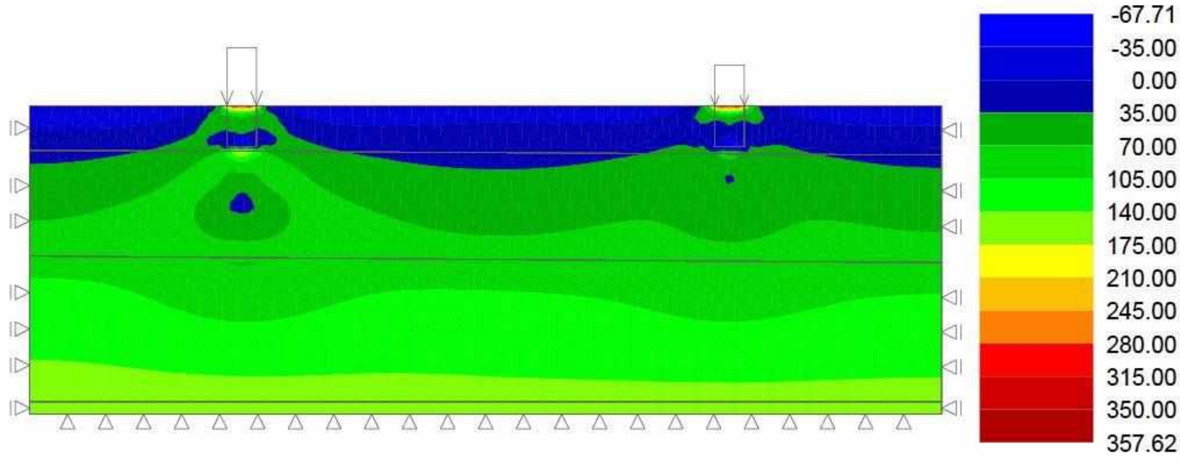


**Figure 51 - Total stress Ϭx, kPa**

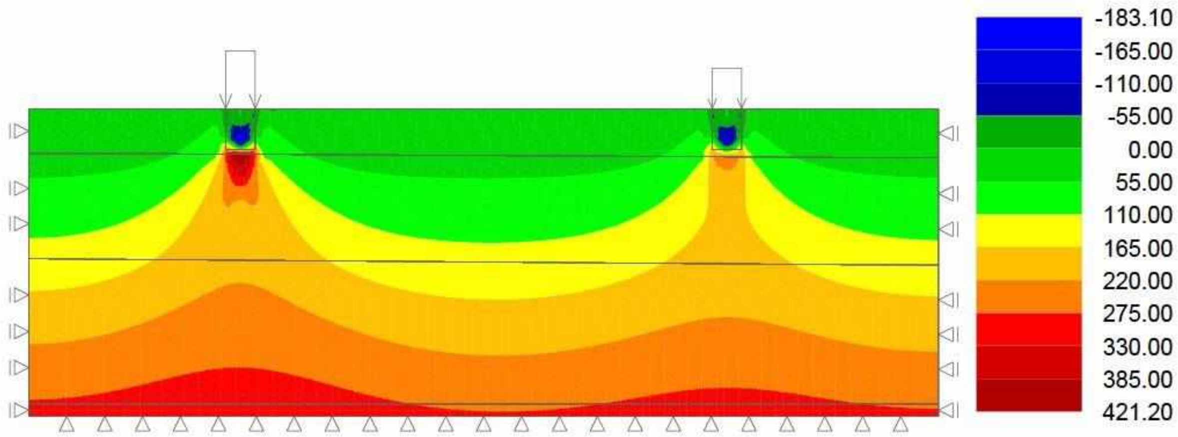


**Figure 52 - Total stress Ϭz, kPa**

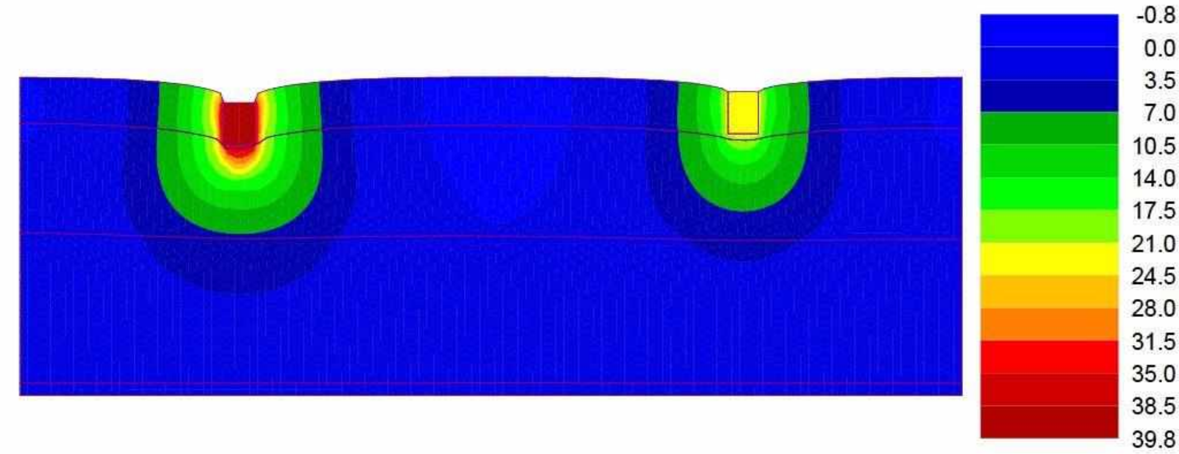


**Figure 53 - Vertical displacement, mm**

The vertical displacement found for tower part is 39.8 mm while for the main part it is 22.9 mm. Concerning the depth of the influence zone found the mentioned vertical displacement, it is about 7.35 m and 5.39 m receptively.

The evolution of plastic deformation zones is shown for both loads in Figure 54.

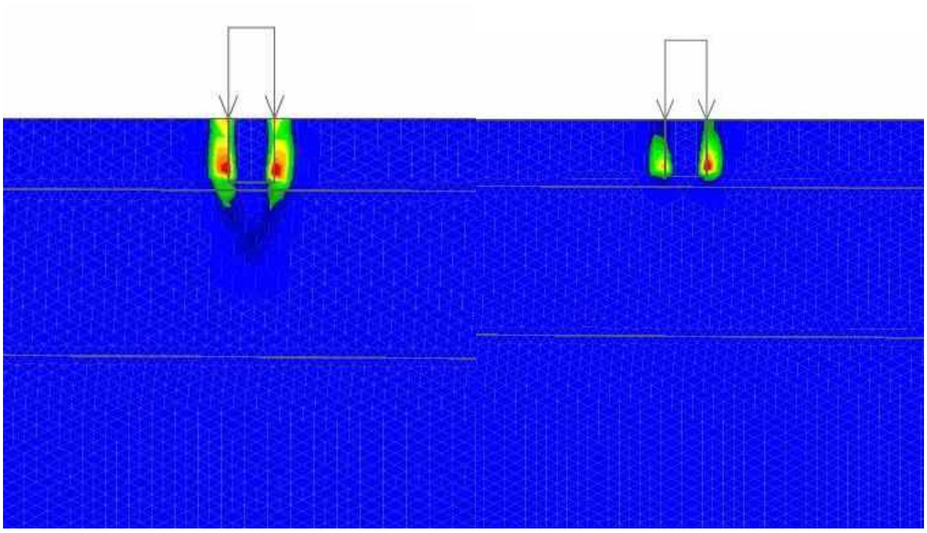

**Figure 54 - Plastic deformation: for load of 700 kN/m(left); for load of 490 kN/m (right)**

The developed plastic zones are similar to the St. Ann Church, however, there is no formation of core under the foundation. This means that the load transmitted to the ground is acceptable and there is no collapse happened due to soil uplift.

### 6.1.4 St. Barbara Church in Otovice

The last church which is analysed in this work is St. Barbara Church in Otovice. It has longitudinal ellipse plan (22 meters long and 15 meters wide) with seven semi-circular chapels. The height of church is 15.5 meters from the floor to the ceiling. The walls thickness is 1.4 meters. The depth of foundation from back part equals to 3 meters while from the front part it is 2.5 meters. The upper structure load was assumed based on Ivana Božulić work and is equal to 480 kN/m. The water level was found at the depth of 3.5 meters; however, it was not considered in the following calculations.

In this case, all calculations were carried out for both boreholes due to the slight deference in the thickness of the layers. Furthermore, the depth of foundation is different hence it affects the value of pre-consolidation pressure.

The modulus of deformation for BH1 received after homogenization equals to 23.25 MPa and for BH2 is 22.99 MPa. These values were used for further evaluations in DEPTH software. Additionally, the effect of pre-consolidations was taken into account by increase of the depth of foundation in twice. However, this value can be a bit overestimated for this church due to a presence of water, consequently it may influence the obtain results. In other words, it will decrease the value of influence zone depth and as a result the vertical displacement as well.

Tables 13-14 show the depth of the influence zone and constants which were used for determination of the vertical displacement. As previously, all calculations were performed for various loads to further development of load-displacement diagrams.

- **Borehole 1 (BH1), foundation depth – 3.0 m**

The homogenized modulus of elasticity for the BH1 was found as 20.79 MPa and the results of calculations can be found in Table 13.

**Table 13 - Calculation results for $E_{hom}$ =20.79 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.208 | 12.021 | 3.925 | 13.09 |
| 480 | 3.948 | 10.169 | 4.357 | 17.42 |
| 550 | 4.584 | 8.434 | 4.878 | 21.41 |

In accordance with the table, the depth of influence zone obtained using the DEPTH software is equal to 3.95 m while the corresponding vertical displacement is 18.90 mm.

- **Borehole 2 (BH2), foundation depth – 2.5 m**

The found equivalent modulus of deformation for the BH1 is very close to the one received for BH2, but the results are different because of varied depth of foundation.

**Table 14 - Calculation results for $E_{hom}$ =20.70 MPa**

| $F_z$ (kN/m) | H (m) | $C_{1W}$ (MN/m$^3$) | $C_{2W}$ (MN/m) | w (mm) |
|---|---|---|---|---|
| 400 | 3.948 | 10.154 | 4.351 | 14.54 |
| 480 | 4.818 | 8.704 | 4.776 | 19.14 |
| 550 | 5.570 | 7.811 | 5.095 | 23.35 |

Depth of influence zone for this church is 4.82 meters and the vertical displacement equals 19.44 mm. A load-displacement diagram was built based on the results above (Figure 55).

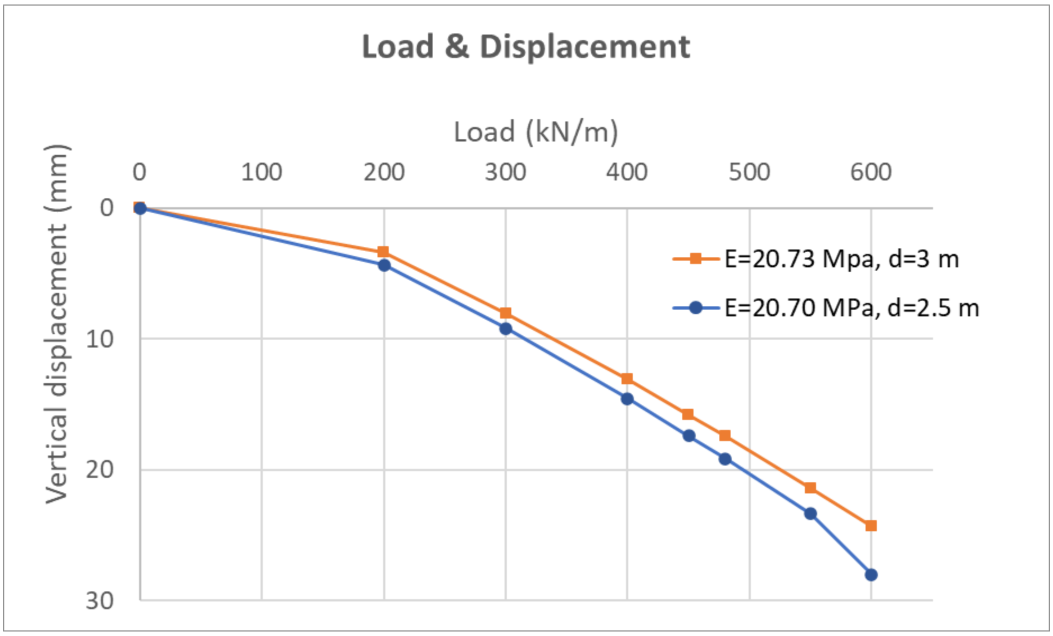


**Figure 55 – Load- Displacement diagram**

As can be seen from the Figure, the curve developed for the foundation depth is equal to 2.5 meters has more cambered character.

After the analytical calculations, the numerical analysis was carried out for further comparison of results. The Figure 56 shows the 2D model with dimensions of 33.8 meters long and 12 meters wide. Stress diagrams are presented in Figures 57-58 meanwhile the vertical displacement is demonstrated in Figure 59.

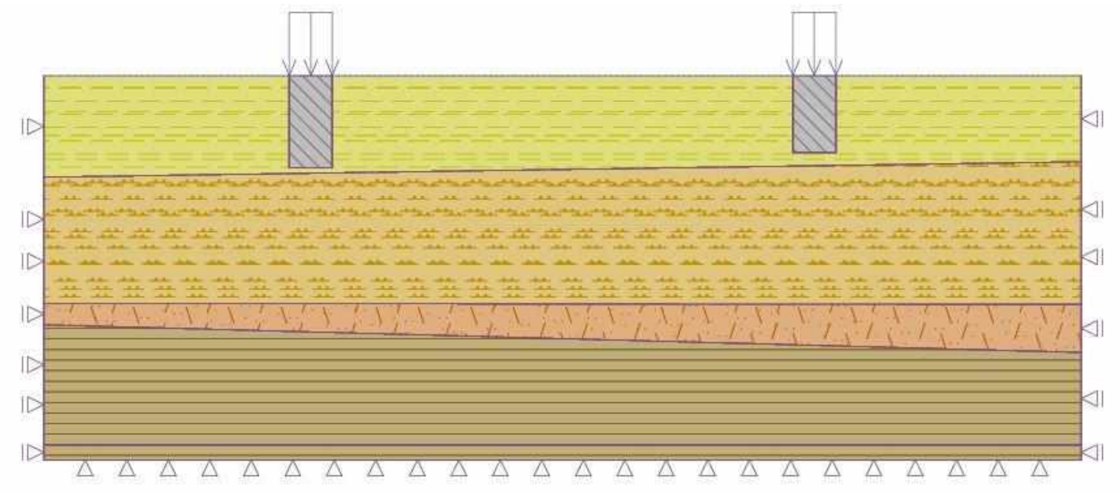

**Figure 56 – 2D Model**

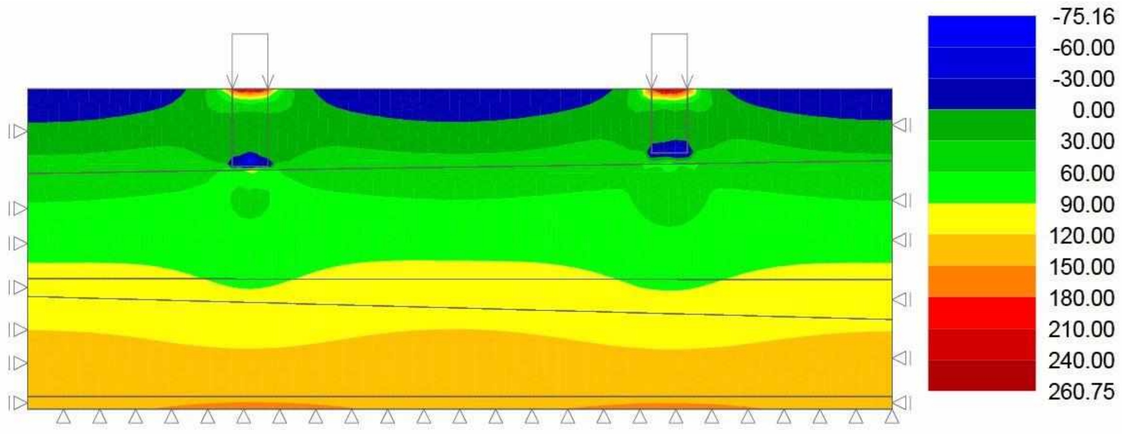


**Figure 57 - Total stress бx, kPa**

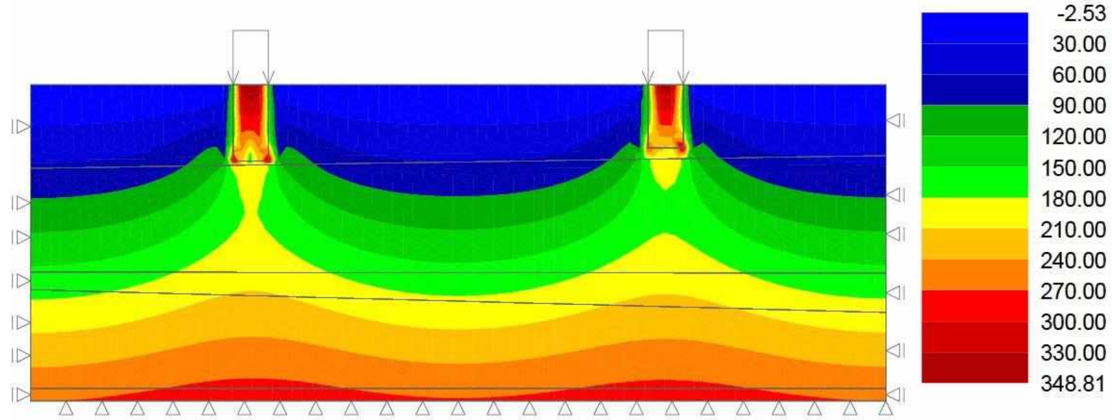


**Figure 58 - Total stress бz, kPa**

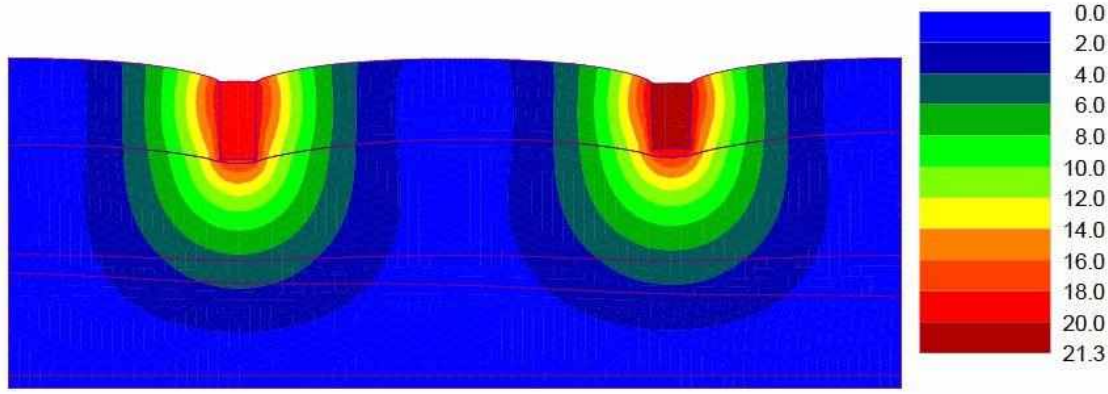


**Figure 59 – Vertical displacement, mm**

The maximum vertical displacement for foundation with depth equals 3.0 meters is found to be 20 mm while for the foundation depth of 2.5 meters, it is 21.3 mm. The corresponding depth of the influence zone is 4.64 and 5.10 meters respectively. Below in the Figure 60 the evolution of plastic deformation is presented.

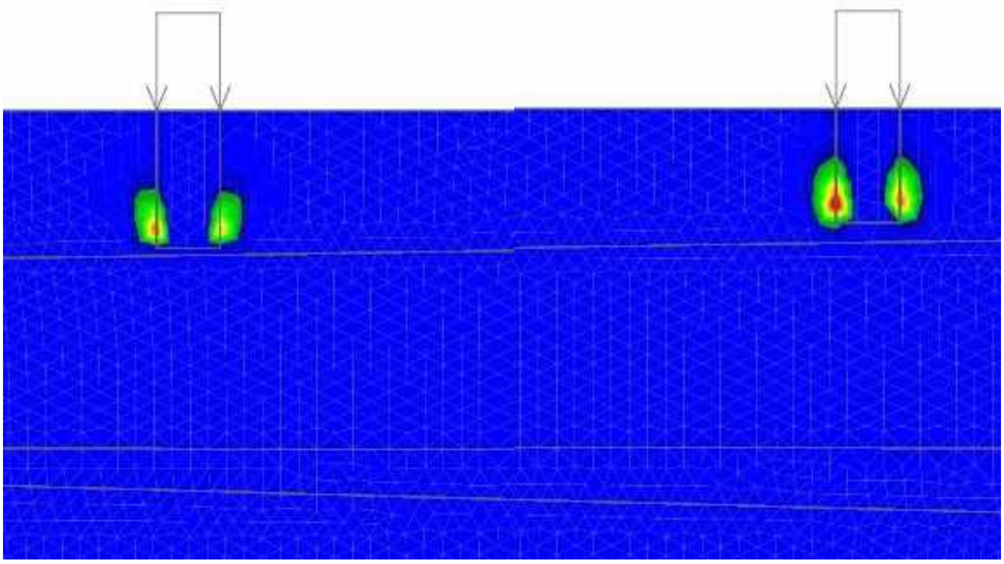

**Figure 60 - Plastic deformation**

As can be noted from the Figure, the plastic zones are smaller than for other churches. This may indicate that the depth of foundation affects the result, and also that the load does not exceed the admissible limit value

# 7. CALCULATIONS ACCORDING TO RUSSIAN CODE

According to the Russian standard SP 22.13330.2016 “Soil bases of buildings and structures”, the calculation of foundation displacements for the mean value of soil pressure under the footing which does not not exceed the bearing capacity of soil should be carried out using a the design scheme in the form of a linearly deformable half-space with a conditional limitation of depth of the influence zone $H_c$.

The foundation settlement can be found by the means of formula below

$$s = \beta \sum_{i=1}^{n} \frac{(\sigma_{zp,i} - \sigma_{z\gamma,i}) h_i}{E_i} + \beta \sum_{i=1}^{n} \frac{\sigma_{z\gamma,i} h_i}{E_{e,i}} \tag{13}$$

where,

β - dimensionless coefficient equal to 0.8

$\sigma_{zp,i}$ – the mean value of the vertical normal stress (hereinafter referred to as the vertical stress) from the external load in the i-th soil layer vertically passing through the center of the base of the foundation, kPa

$h_i$ - thickness of the i-th layer of soil, cm, taken as not more than 0.4 of the width of the foundation.

$E_i$ - modulus of deformation of the i-th soil layer along the primary loading path, kPa.

$\sigma_{zy.i}$ - the mean value of the vertical stress in the i-th layer of the soil vertically passing through the center of the base of the foundation, from the own weight of the soil chosen during the excavation of the foundation pit, kPa

$E_{e,i}$ - deformation modulus of the i-th soil layer along the secondary loading path, kPa.

n - the number of layers into which the depth of the influence zone is divided

With regard to the above mentioned, the distribution of vertical stresses along the depth of the foundation is taken in accordance with the diagram shown in Figure 61.

It should be noted that during the construction of structures in the excavated pit, the following three values of vertical stresses should be distinguished:

- $\sigma_{zg}$ - from the self-weight of soil before the construction
- $\sigma_{zu}$ - after excavation of pit
- $\sigma_z$ - after construction of structure

Furthermore, If there is no test results of the modulus of deformation $E_{e,i}$ for the buildings of second and third category of responsibility, it is allowed to use the following relation $E_{e,i}$ = 5$E_i$. Another assumption, if there is a layer of soil with a deformation modulus E> 100 MPa within the depth $H_c$, which was found according to the above conditions, the compressible thickness is allowed to be taken up to the top of this soil.

When calculating the settlement of foundations constructed in pits less than 5 meters deep, it is allowed to ignore the second term in the formula.

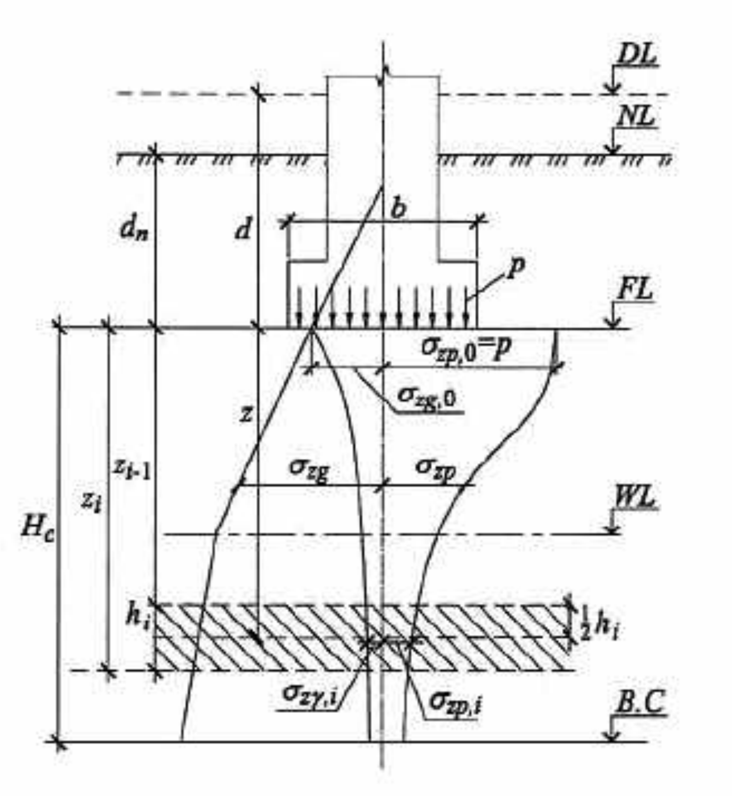


**Figure 61 - Diagram of the distribution of vertical stresses in a linearly deformed half-space (SP22.13330.2016)**

<u>Legend:</u>

DL - layout mark; NL - elevation of natural relief surface. FL - elevation of the base of the foundation.

WL - groundwater level; B.C - the lower boundary of the influence zone depth.

d and $d_n$ - the depth of the foundation from the level of planning and surface of the natural relief, respectively; b is the width of the foundation; p is the average pressure under the base of the foundation.

$σ_{zg}$ and $σ_{zg,0}$ - vertical stress due to the self-weight of soil at a depth z from the base of the foundation and at the level of the base; $σ_{zp}$ and $σ_{zp,0}$ - the vertical stress from the external load at a depth z from the foot of the foundation and at the level of the foot; $σ_{zγ}$ - vertical stress due to the own weight of soil excavated in the pit in the middle of the i-th layer at a depth z from the base of the foundation; $H_c$ - the depth of influence zone.

Vertical stresses from an external load $σ_{zp}= σ_z - σ_{zu}$ depend on the size, shape and depth of the foundation, the distribution of soil pressure along its bottom and the properties of soils. For rectangular, round and strip foundations, the values of $б_{zp}$, kPa, at a depth z from the bottom of foundation vertically passing through the center of the base, are calculated in accordance with the equation

$$\sigma_{zp} = \alpha p \tag{14}$$

where α is a coefficient taken from the Table 5.8 SP 22.13330.2016 and depending on the relative depth ξ equal to 2z/b;

*p* is an average pressure under the foundation bottom, kPa.

The vertical stress from the self-weight of the soil at the level of the base of the foundation $б_{zɣ} = б_{zg} - б_{zu}$, kPa, at a depth z from the base of rectangular, round and strip foundations, is calculated by the formula

$$\sigma_{z\gamma} = \alpha \sigma_{zg,0} \tag{15}$$

where α is the same as in the previous formula

$б_{zg,0}$ is a vertical stress from the self-weight of soil at the level of foundation, kPa and is equal to

$$\sigma_{zg,0} = \gamma' d \tag{16}$$

γ is a density of soil, $kN/m^3$, d and dn are mentioned above.

The lower boundary of influence zone is taken at a depth z=$H_c$, where the condition $б_{zp}$=0.5$б_{zg}$. Nonetheless, the depth of the influence zone should not be less than b/2 for b ≤ 10 m, (4+0.1b) for 10 <b ≤ 60 m and 10 m for b > 60 m.

All calculations were carried out in a simple software Foundation 13.3 developed by Basegroup.su. The "Foundation" software performs calculations of underground structures. All calculation theories are taken in conformity with existing Standards, as well as manuals and annexes to them.

After performing calculations according to the Russian code, the results were tabulated below (Table 15). The water level was not considered in those calculations despite the fact that it has an influence on the result.

**Table 15 - Calculation results according to the new Russian code**

| | St. Ann Church | | St. Jacob church | St. Barbara Church | | All Saints Church | |
|---|---|---|---|---|---|---|---|
| Foundation depth (m) | 1.8 | | 1.5 | 2.5 | 3.0 | 1.65 | |
| Load (kN/m) | 460 | 730 | 440 | 480 | | 490 | 700 |
| Depth of influence zone (m) | 4.8 | 6.2 | 4.8 | 4.7 | 4.5 | 5 | 6.1 |
| Settlement (mm) | 29.56 | 52.56 | 29.19 | 29.98 | 29.48 | 30.28 | 45.74 |

As it can be seen the average value of depth of the influence zone is about 5 meters for the load in range 440-490 kN/m and about 6 meters for the load in interval 700-730 kN/m. The mean settlement for minimum and maximum load equals approximately 30 mm and 50 mm respectively.

Although it is worth nothing that a few years ago the Russian Standard was updated and the relation for low boundary of the influence zone depth was changed from 20% to 50%. For this reason, all calculations were also carried out in correspondence with the old version of Russian standard to see how it may affect the results. The determined results are summarized in the following Table 16.

**Table 16 - Calculation results according to the old version of Russian code**

| | St. Ann Church | | St. Jacob church | St. Barbara Church | | All Saints Church | |
|---|---|---|---|---|---|---|---|
| Foundation depth (m) | 1.8 | | 1.5 | 2.5 | 3.0 | 1.65 | |
| Load (kN/m) | 460 | 730 | 440 | 480 | | 490 | 700 |
| Depth of influence zone (m) | 8.6 | 14.23 | 8.5 | 6.8 | 6.4 | 9.05 | 13.05 |
| Settlement (mm) | 35.75 | 65.1 | 34.5 | 34.7 | 34.4 | 35.7 | 54.9 |

Based on the results, it can be noted that the value of vertical displacement has raised in almost twice for the depth of influence zone and on 5-10 mm for the vertical displacement. However, such a significant increase in the values of the influence zone depth did not lead to the same growth in settlement.

It should be mentioned that loads used in all calculations in accordance with Russian code were calculated with respect to Eurocode. It is evident that load factors in both standards differ from each other. For the reason given earlier, the design load would also be different. Furthermore, soils properties were assumed on the European recommendations. Usually, soil properties for Service

Limit State (SLS) is taken with different safety factor in Russia thereby considering inaccuracy of values and giving an additional safety.

Based on all reasons mentioned above, it can be concluded that if calculations according to the Russian standards were performed completely with respect to other Russian codes, the result of calculations may be different from the one obtained.

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## 8. COMPARISON OF RESULTS AND CONCLUSIONS

Analysis of vertical displacements as well as depth of influence zone is an important part of building calculations, since the settlement, especially the differential settlement, is a common cause of damage particularly cracks in buildings. Additionally, the depth of influence zone plays a significant role in estimation of settlement. That is why it is necessary to determine them during building analysis.

Therefore, this work was focused on calculation of depth of influence zone and vertical displacement using the example of Broumov group of churches. In order to make the comparison more demonstrative, all obtained results were summarized in the Table 17 below.

**Table 17 - Final calculation results**

| Name of church | | Depth of foundation (m) | GEO5 FEM | Depth software | Russian code (new) | Russian code (old) |
|---|---|---|---|---|---|---|
| St Jacob Church | S, mm (*440 kN/m*) | 1.5 | 24.90 | 18.9 | 29.19 | 34.5 |
| | H, m | | 5.90 | 4.96 | 4.80 | 8.50 |
| St. Ann Church | S, mm (*460 kN/m*) | 1.8 | 24.50 | 18.00 | 29.56 | 35.75 |
| | H, m | | 5.85 | 4.29 | 4.80 | 8.60 |
| | S, mm (*730 kN/m*) | | 49.10 | 34.76 | 52.56 | 65.1 |
| | H, m | | 7.56 | 6.94 | 6.20 | 14.23 |
| All Saints Church | S, mm (*490 kN/m*) | 1.65 | 22.90 | 19.90 | 30.28 | 35.7 |
| | H, m | | 5.39 | 4.90 | 5.00 | 9.05 |
| | S, mm (*700 kN/m*) | | 39.80 | 31.10 | 45.74 | 54.90 |
| | H, m | | 7.35 | 7.09 | 6.10 | 13.05 |
| St. Barbara Church | S, mm (*480 kN/m*) | 2.5 | 21.30 | 19.14 | 29.98 | 34.70 |
| | H, m | | 5.10 | 4.82 | 4.70 | 6.80 |
| | S, mm (*480 kN/m*) | 3.0 | 20.00 | 17.42 | 29.48 | 34.40 |
| | H, m | | 4.64 | 3.95 | 4.50 | 6.40 |

The maximum and minimum values for vertical displacement and depth of influence zone are highlighted. The least value of settlement (light blue) was found by Depth software whereas the highest value (blue) was calculated according to the old version of the Russian code. As regards the influence zone depth, the minimum values (light pink) were determined using the updated version of the Russian code while the maximum values (pink) were obtained by means of the old Russian code. So, it can be noted that the old version of the Russian code provides the maximum values for both vertical displacement and depth of influence zone. It can be assumed that the old version of the Russian standard overestimates the values because it uses a more conservative approach.

It is not known for certain why the Russian standard was changed, but it can be suggested that after years of monitoring of foundation settlement for new constructions it was decided to limit the depth of influence zone earlier to have the results closer to reality.

In this work it was not possible to compare which method provides a better result due to the absence of monitoring results. For this reason, the methods were compared with each other only. It is generally accepted that numerical analysis provides quite reliable result. Therefore, all analytical solutions are compared with it. Both analytical solutions (according to the updated version of the Russian code and by means of DEPTH software) have an average difference about 5-6 mm for the vertical displacement and about 1 meter for the depth of influence zone. The values obtained by DEPTH software is generally smaller than the numerical solution provided. On the other hand, new Russian code provides higher value of settlement and smaller value of depth of influence zone. Although, both of them have relatively good convergence in results.

As it was mentioned above, the construction costs were partially paid by local parishes, so the churches were quite simple. However, St. Barbara Church has more complex plan as well as a larger and deeper foundation indicating that the amount of money spent on its construction was higher. Moreover, the investigations showed better state of the church despite the presence of ground waters. It is also worth noting that the construction of deeper foundation ensures its non-freezing condition. After analysis of the results, it is evident, that a better quality of foundation built for St. Barbara Church allowed to decrease the value of vertical displacement as well as depth of influence zone. Moreover, the smaller size of plastic zone observed, indicates a greater reserve of bearing capacity of soil.

# 9. DISSCUSSION

The foundations of historical structures are mostly built from natural stone. This stone is often identical to subsoil due to usage of material from surrounding deposits. Natural weathering of stone is inevitable; however, it can be speeded up by some reasons.

The main cause leading to stone deterioration is a presence of water. This problem triggers other causes leading to destruction of the material.

Most frequently, water is supplied in the form of atmospheric precipitation being further infiltrated into the ground. And if the building does not have a proper drainage system, both on the ground and on the roof, this leads to the surface soaking. Constant presence of water creates a favourable environment for development of biological activity. Another consequence of water presence in the material is a faster decay due to thaw-freeze process. Frost weathering happens due to the stress resulting from growth of ice crystals within the freezing pore water. Moreover, if the water or stone contain chemical components, this leads to the process of crystallization and subsequent destruction of the material.

As it was observed during the inspections performed in previous years, that all churches suffered from the moisture and lack of proper drainage system. So, this problem is often found in historical buildings.

Another reason of degradation is lack of maintenance. All churches were found in a very poor technical condition due to the fact that no measures were taken to restore them or at least to maintain their condition. For this purpose, timely and sufficient maintenance could help to avoid serious damage and, as a result, to reduce restoration costs in the future.

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## 10. RECOMMENDATIONS

The following general recommendations will be divided into two parts. The first part will be related to the analysis of buildings while the second part will be focused on the possible remediation solutions. All recommendations will be suggested on the basis of the previous discussion, common problems existing in historical structures as well as the investigations carried out earlier by other people, since the situation did not allow to perform the inspection directly.

First of all, before the analysis of historical building, proper investigations should be carried out. They should start with soil survey including in-situ tests as well as laboratory works. All properties of soil necessary for the calculations should be found based on test results. Moreover, the research of foundation material should be carried out. All obtained data should be sufficient to perform the analysis with the selected material models, otherwise it would be difficult to achieve an adequate result.

Unfortunately, sometimes it is difficult to conduct of a soil and foundation survey due to their location under surface and the limitation of funding. However, incomplete data about soil and foundation conditions may lead to the future problems such as additional settlement. So, this factor should be taken into account during the restoration.

The long-term monitoring system may help to understand the reasons of degradation processes as well as to control the development of vertical displacement of foundations. Knowing the causes, it is much earlier to select the necessary solutions. Thuswise it can minimize the amount of money spend on preservation.

Regarding the rehabilitation of the churches, some general recommendations can be proposed.
In order to decrease the amount of water and moisture, an adequate drainage system should be installed, or the old system could be replaced. One system for dewatering on the ground and the second one to drain water from the roof. It may help to stop the degradation process and perform restoration. Another possible solution is a ventilation system which could help to prevent water retention inside of the material. Furthermore, a monitoring system to control the moisture level could be installed in future. If the humidity level rises, the appropriate immediate measures could be taken to reduce it.

Another widespread problem is evolution of cracks in the structure. Such cracks should be carefully analysed in order to find a reason of their formation, otherwise it would be impossible to stop their further expansion. As it was mentioned in Section 8, one of the possible reasons of crack formation is the foundation settlement.

The problem of differential settlement that was also identified during the inspections may be resolved by soil stabilization, for example, by using different strengthening techniques such as jet-grouting. However, firstly, the reasons of settlement evolution should be found.

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