1. INTRODUCTION

The case study developed in this thesis is oriented to the analysis of the Church of the Sacred Heart, “kostel Nejsvětějšího Srdce Páně”, in Prague. This building is located in the neighbourhood of Vinhorady - Prague 3 - in the Jiřího z Poděbrad square and its location is part of the composition of public space connected in perpendicular direction that marks this neighbourhood.

This object of the case study is considered to be one of the most important example of the 20th century religious building in the Czech Republic. It was designed by a Slovenian architect Jože Plečnik who is also known in the capital for the modifications of the gardens of the Prague Castel in 1920 at the invitation of President Masaryk.

The Church is located almost in the centre of the square and it is aligned in its long side to the West-East axis. As shown in the Annex 1 Location of the Case Study “Church of the Sacred Heart”, the public open space around the Church, defined by residential buildings of the 19th century, shows a remarkable example of urban planning. In the west side, directly from Mánesova street, a line of trees and benches defines the path to the entrance of the Church. On the north side the green garden is separated at the tower level for creating another access to the building; furthermore, the corner of the square with Slavíková street is used three days a week as an open market stressing the social important that the square assumed for the neighbourhood. In the south line is it possible to find a secondary path lined with trees that, starting with a fountain, develops parallel to the Church and that almost matches with the Jiřího z Poděbrad metro, opened in the 1980, which entrance is located in the corner with Uvodámy street. The back side of the building, facing East, is characterized by a private garden used also as a parking for the people of the Church; beyond this, a rectangular area defined by the grass on the surface surrounds two ramps which represents the accesses to an underground parking.

Another important element that marks the landscape not only of this area but of the entire city is the Žižkov television tower completed in 1992.
Inspections of the Church of the Sacred Heart that were carried out in recent years ([2],[3]) have revealed that the building exhibits many structural and non structural faults. Against this background and considering the above-discussed importance of the building, the main goals of the present thesis were set as follows:

1. to perform an additional structural evaluation of the building;
2. based on item 1, select one of the most significant damage and perform a detailed analysis of its cause.

The procedure followed in this thesis for the evaluation of this historical building is synthetically explained in Fig 7. The characterization of the object of the case study will start with the historical survey in order to be able to understand the process that brought it to its actual shape including all the modifications or damage that the building had to experience during its lifetime. The documentation available from previous survey will be then analyzed and, on the other hand, the history of the architect and of the construction works will be investigated obtaining from that important information related to the cultural value of this building. Moreover the research will try to find historical pictures of the building under construction which can contribute to define a general timeline of the building construction phases, of the technologies applied and of the typology of materials used.

After this first phase the building was investigated throughout the visual inspection. A photographic survey was produced during different surveys performed in the majority of the building with various experts groups. In order to ensure the complete description of the structure this phase was subdivided in further topics:
geometrical survey, technology identification, materials characterization and damage identification. Throughout these phases it was possible evaluate the condition of the overall building and of its different parts. This diagnosis permits to identify the areas that need an adequate maintenance plan and the ones where a deeper investigation is necessary due to the presence of more severe damage. As will be explained later the most significant structural damage pattern was identified in the tower where therefore further steps were considered. In particular, our goal is to validate the hypothesis that these faults result from thermal effects. Due to the aim to investigate the thermal behaviour of the massive walls of the tower a monitoring plan for detecting the thermal and physical conditions should be performed. Due to the short term available and to the fact that a monitoring of these conditions becomes representative after at least one year, a Building Performance Simulation (BPS) program, which has at its disposal a database of typical-year local climatic data, was employed instead of monitoring. The enormous amount of data resulted from the BPS program was then evaluated and considerations about long and short term behaviour of the walls of the tower were taken into account in order to define the data to input in the thermo-mechanical analysis.

**Fig 7 Procedure followed for the analysis of the case study**
Analysis of the faults in the Church of the Sacred Heart in Prague
2. HISTORICAL RESEARCH

The historical research was approached following different topics. From one side the development of the Church was studied together with the main events of the life of its Author. This analysis allows to define the reasons behind the final shape of this building and it permits to understand its value and importance. The process of construction of the Church as well as Plečnik life are strongly influenced by the contemporary historical events and architectonical evolution.

![Diagram showing the development of the Church and main events in Jože Plečnik life.](image)

**Fig 8 Development of the construction of the Church and of the main events in Jože Plečnik life related with the architectonical evolution between the 19th and 20th century**

2.1 Architectonical evolution between the 19th and the 20th century

As the development of the project of the Church covers a period between the second half of the 19th century and the first part of the 20th century it is important to define the main architectural and historical happenings also related to the Vinhorady neighbourhood. In this way it is also possible to understand the importance given to the realization of this Church, that now is waiting to be inserted in the UNESCO Heritage List [4].

The neighbourhoods of Žižkov and Vinhorady are two of the residential districts of Prague developed during the second half of the 19th century. In this period the city experienced a significant industrial growth that contributes to a need for accommodations in the immediate future; on the other hand several public building (institutions, schools, museum, Churches, ..) were built in order to satisfy the new population of the city. The neo-renaissance style was selected as national style although an eclectic mix of styles characterizes the...
architecture of this period (neo-romanesque, neo-gothic, ..). The art movement of Secession characterized the architecture of the city during the first decade of the 20th century. This movement spreads from Germany and Austria-Hungary with its revolution against the traditional art movement. Several buildings were built among which the residential buildings and the community house in Prague were symbols of the art belonging to the middle class and not only to rich people. Even considering the freedom of style and the union of different arts that characterize this period, the secessionist buildings can be identified by their symmetric façade where the central part usually stood out through decorative elements (as arches, curve shape elements, bigger windows, ..) or through the elevation of the two edges of the façade itself. The use of reinforced concrete and steel was investigated in these buildings with always the attempt to divide the public space from the private one [5].

In the fast changes of the architectural evolution during the 20th century, the second decade was characterized by the cubist architecture. The movement, developed in the art with principal exponent Picasso, was used in the architecture of this period as new form of expression. The faceting of the paints were transfer in architecture using simple geometrical elements created with reinforced concrete or steel that give dynamite to the buildings. The progress in the use of these material can be seen through the presence of big openings often continuous through all the façades.

The first world war (1914-1918) brought a sharp-cut in the history with the end of the Hapsburg Empire supremacy and the creation of the Czechoslovakia state. Prague was selected as capital and its castle was the residence of the president Tomáš Masaryk. As mentioned before the president itself asked to Jože Plečník some of the modifications of the castle. The important increase of population as well as the feeling of insufficient building for the new functions of the state generated a new productive period for the architecture. Several new districts were built as the nowadays important neighbourhood of Dejvická. On one hand the architecture was characterized by the recurs of the historical forms with frequent connections to classicism; on the other hand a new architectonic style named functionalism developed. This new style, without decorations, had to represent and to show the technological and industrial evolutions of that time. Therefore, glass wall in the façades were supported by sophisticated reinforced concrete frames. Functionalism will be one of the most important, if not exclusive, styles in the Czechoslovakia during the 20th century.

2.2 Development of the project of the Church of the Sacred Heart

The main dates linked to the development of the Church are illustrated below in a chronological order. These include the main events in the life of the architect Jože Plečník and the main happenings that permit the beginning of the building of the Church in the 1928.

1893 Consecration of St. Ludmila Church in Náměstí Miru, Vinhorady.

This was the first Church built in the Vinohrady in this period from the architect Josef Mocker. In the fast economical development that characterized that period, the neighbourhood realizes the need of more spaces for Catholics.
1894-98 *During this period the architect Jože Plečnik attended the Vienna Academy of Fine Arts.*

This is the last phase of Plečnik training, after working in his father joiner's shop in Ljubljana. During this time he will be strongly supported by Otto Wagner, director of the academy. Thanks to his professor, in line with the Secessionist movement, he experienced the simplicity that was at the base of the new concept of architecture; this was expressed by the shapes and the construction itself. Also the decoration were limited and used with a functional purpose. On the other hand, Plečnik attention was focused on the ancient Greek art, considered as the only style from the past matching simplicity with symbolism, order and harmony. The contact with this architecture was even stronger the year after his dissertation when he travelled with a scholarship around Italy.

These characteristics together with the strong Christian faith - always kept alive by his family in Ljubljana - and the aim to find the origin and identity of Slavism architecture and culture would have represented the engine of its future works.

1905 *The 24th Vienna Secession exhibition.*

Beuron Benedictine monks-artists were invited to exhibit their works in Vienna. Plečnik ideas were well reflected in the elementary simplicity and purity investigated by this religious group in the regular drawing and symbols of Early Christian. This thought belonging to the monks was maintained alive in Plečnik works during the time, and was alto taken into account in the later addiction inserted in the building as the decoration of the windows in the Church area.

"..we have been chosen by God… but if we are to create works of art, we must know that we are not artists – but that we should, in suffering, in our search for what is beautiful and good, come as close as possible to God – to the ideal of justice, and foster good people – good, honest and as immaculate as possible. Everything else will come of its own accord.."

..wrote by Plečnik to his friend Jan Kotěra in the 1908 [4]

1909 *Plečnik left Vienna to start a professorship at the Academy of Applied Arts in Prague.*

As explained above, Plečnik art originates from different lines, This makes his style unique for the time. For this reason at that time he left Vienna and its Secession movement that was not completely representing his architectonical point of view. In Prague, his devoted students became sensitive on Greek art, on the craftsmanship and on all the ideas at the base of Plečnik projects (simplicity, functionality, monumentality, ..).
March 1914  “Society for building a Second Catholic Church in Vinhorady”.

During this period the strong need of a religious place produces the foundation, during the March of the 1910, of a new parish with the chapel of St. Aloise as a temporary solution for the liturgy. This was located in the Jiřího y Poděbrad square and was consecrated the last day of the 1913.

The aim of building a new Catholic Church in Vinhorady culminates in the March of 1914 with the creation of a “Society for building a Second Catholic Church in Vinhorady”.

1914-1918  First World War

28 October 1918  The sovereign state of Czechoslovakia born.

This political change with the figure of Tomáš Garrigue Masaryk subsequently involves a drastic change in the culture and dimensions of the country. As new capital, Prague experienced a fast increase of population. Furthermore the Prague castle became the home of the president. It is in this period that Masaryk applied several interventions in the castle area, some of which were assigned to Jože Plečnik (1920).

The lack of residential areas was fulfil by the new country with several architectural competitions attended also by Plečnik’s students with considerable success.

The activity of the “Society for building a Second Catholic Church in Vinhorady”, blocked before starting by the WWI, was finally free to develop his task. In this complicated period after the war in a newborn country the Society saw in the figure of Plečnik a professional authoritarian figure already estimated by the society and with a strong catholic faith able to drive him to the right shape of the Church.

August 1920-1922  Plečnik left Prague for going to teach in the Ljubljana School of Architecture in his hometown of Ljubljana.

After the request from the architect Ivan Vurnik in the 1920 Plečnik accepted to become a professor in the technical college in his town starting in the 1922. In this way he would have moved two years later to Ljubljana where he remained up to his death. During these years he accepted the prestigious commission for remodelling the “Paradise Garden” and the “Rampart Garden” in the Prague castle.

February 1919  The Society of Czech architects proposes Plečnik project for the Church

Just before the announcement of the competition for the realization of the Church of the Sacred Heart the society of Czech architects suggests Plečnik as architect to the Planning Committee.
However, as the competition could not be cancelled they asked Plečnik to submit his project outside of the competition without receiving a positive response.

April 1919 The “Society for building a Second Catholic Church in Vinhorady” announced the competition.

The projects should consider the space for 50’000 parishioners as well as the presence of an organ, a choir and five altars; secondary spaces as sacristy, baptistery and presbytery should have been also included. Furthermore, the design of the Church should have included the urban planning of the Jiřího z Poděbrad square.

Over 31 projects were submitted considering also the excellent ones of the Plečnik students. Unfortunately the project was stopped: legal problems regarding the use of the land appear and an apathy-negative mood spread related to the presence of a new Catholic Church in Vinhorady.

Even if Plečnik always preferred to not be active in the developments concerning the Church, his decision of writing a letter to Alexandre Tilt marked on one hand the beginning of a friendship and on the other hand a rich soil for the development of the Church project.

June 1921 The first project of Plečnik.

In the first project the architect selects to link the Church to the school chapel of St. Aloysius in the square. Designed with circular shape the Church should have been surrounded by residential buildings and accessible from several sides.

March 1923 The second project of Plečnik was submitted to the Archiepiscopal Consistory.

The Society asks to Plečnik to modify his project as asked in the original competition of 1919, not connecting it with any building around. Plečnik project results in the design of different buildings, a Church, a school ad a rectory. The religious building would have been as tall as the residential area around and surrounded by columns as a doric temple with a venetian-style bell tower.

The new design was accepted but two main problems arose: the project was very economically demanding and the State Regulatory Commission did not agree to build the entire site. While the first problem should have been solved by a reduction of the project, for the second one Masaryk intervene in the Church favour, with the help of a legal opinion of the faculty.

1925 The third project of Plečnik.

A reduced less expensive project was proposed considering only the Church without its greek appearance. The Church lost its greek style in favour of the Czech cubism that characterized the
second decade of the 20th century. Understanding the lack of money of that moment his idea was to build a Church during the time following the economical budget that the parish would have found in the meanwhile. The project turned to be simpler with just a room where altars would have been spread in its perimeter. The height of the Church was controlled in order to reduce the money needed for heating or for a better acoustic environment.

Also the position of the tower was changed and attached to a side of the building. However Plečník sent his project to Tilt expressing in his letter his dissatisfaction and his recommendation for finding a better professional for the new project.

1928 The last plan that solves the problems with the land was sent and approved.

1928-32 The working for the construction of the Church started.

The works started just one year after the consecration of the first stone actually due to the poor load bearing capacity of the ground which required a reinforced concrete slab foundation involving a significant increase in the budget needed for the building.

26 August 1929 The excavating for the foundation of the main facade wall started. The design depth of 5 m have been changed to 7 m in order to arrive to the rock soil. Even under the tower digging works were carried out in order to build a reinforce concrete slab.

May 1930 The Church walls were built up to 1 m.

June 1930 The Church walls were built up to 5 m. Also the choir for the organ was built.

September 1930 Part of the tower was already built. In the same month the steel trusses of the roof of the Church start to be arranged in the floor and then lifted up on the small cantilevered concrete supports.

October 1930 The masonry of the tower arrived at the point of the clock opening.

November 1930 The tower was concluded with the trusses and a temporary layer closing the roof. On the other hand the roof of the Church was finished including the copper layer.

1931 At the beginning of the year the cold winter prevent the continuation of the works, just small interior works could be done. Then all the missing parts in the exterior (copper, ..) as well as in the interior (coffer, ..) were positioned.

8 May 1932 The Church of the sacred Heart was consecrated.

8 February 2010 The Church of the Sacred Heart was declared a Czech National Heritage site.
2.3 Inspiration of the Author

The unusual and innovative shape that Plečnik selected for the Church was an attempt to renovate the Catholic faith in the society of the new Czechoslovakia; this religion was deeply associated to the Hapsburg Empire and then after 1918 was considered part of its unwanted rules.

The Architect was believing in a modern idea of Church as a place where devotes can go for the liturgy but where they can also meet and share during the day. This philosophy inspired his project that pretends to be the paraphrase of Noah’s Ark: for analogy the Church should receive the devotes and protect them during the storm sea that was shaking the religious faith at that time. In this way, the main body of the Church represents the ship and the width massive tower should represent the most with blowing sails. In the interior also the coffered ceiling should remember the under deck of a ship [6].

In conclusion, Plečnik work cannot be easily classified in the different architectures of the beginning of 20th century. His personal style, developed during his life, was combining Czech elements from his Slavic belonging with element remembering the age of Charles IV and the Early Christian liturgy and symbolism discovered with the monks during the exposition of 1905. Also the ancient Greek architecture was predominant in his projects: from the original plan the Church should have been surrounded by a line of columns that for economical problem was never realized. By the way the influence of Greek style can still be appreciated by the simplicity of shapes usually rhythmically repeated as well as by the entrance basement with the three staircases and entrances that give monumentality to the main façade.

Considering the criticism and the several problems that were affecting the Church during its design, Plečnik was able to develop a strongly innovative project that was not matching with the art of that time. However, he decided to show his structural ability in the tower area building the two big circular openings, with the clock facing the castle and the ramp that follows the perimeter of the tower up to the bell space. In this part it is clear the attempt to demonstrate to be able to manage the new materials innovations that were shown off by the functionalism even if these were not characterizing its project.
Analysis of the faults in the Church of the Sacred Heart in Prague
3. VISUAL INSPECTION

After the historical survey the visual inspection was performed considering the following four main tasks: the geometry validation of the plans, the technology identification, the characterization of the materials and the damage identification of the building. In order to investigate these topics several surveys were performed during the month of April with different teams as shown in the table below. Moreover the documentation about previous surveys was analyzed in this phase and validated during this work where the visual inspection was performed. In the other areas of the building that were not accessible during the inspection days the considerations done in the latest survey were assumed. In particular, during the last survey available in literature [2] an important testing phase was performed throughout which it was possible to characterize the properties of some of the construction’s materials and to investigate the technologies of the different structural and non structural elements.

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Table 1 Schedule of the visual inspection survey

3.1 Geometrical survey

The Church of the Sacred Heart is a regular building with simple geometry developed in plan and in the height. It is mainly composed by a rectangular volume that follows the lines of the buildings around and by a tower. This volume can be subdivided in three main areas: the west religious area, the east administrative area and the volume of the tower. The bottom part of the tower represents the connection between the east and west areas.
In order to characterize geometrically the building in the short time available the measurements from the previous documentation [7] were considered. The main dimensions and the ones significant for the numerical model were then verified during the inspection days (Annex 2 - Visual inspection (geometry [7], materials, technology [3]), tests location [3]).

The building is symmetric respect to the E-W axis with main facade in the west side. Moreover, the symmetry is proposed also in the height with the use of the same material composition and openings location for the Church and for the tower. A first analysis was carried out considering the different usage of the building and the location of the access to the spaces as shown in the Annex 3 - Usage of the spaces of the building.

The building presents six entrances of which three are in the main entrance of the Church, one is in the administrative area on the east façade and two symmetric lateral entrances in the tower volume.

It is possible, taking also into account the different usage of the spaces, to divide the building in tree sub volumes: the religious part identified with the west volume, the east area with administrative spaces and the tower volume characterized by connection spaces in the lower part.

![Diagram of the building](image)

The religious part of the building, concentrated in the west area, occupies the biggest volume. The Church area is around 26x38 m from inside and 28x40 m from outside; it occupies the main rectangular space with an height of 17 m from outside. The main access to the Church is in the west façade and it is composed by three main entrances that are elevated from the ground level through a floor accessible by three staircases, one central and two in the sides. The thickness of the wall change from 1.05 m to 75 cm due to the rhythmical change of the wall in the inside.

![Dimensions of the lateral walls](image)

Up to 10 m then the walls continue regular with simple golden cross decorations. Above all, a line of windows mark the final part of the interior of the Church; a door located on the left of the east side wall...
(facing the main altar) represents the entrance to a small path at the windows level that follows the perimeter of the Church.

Fig 12 View of the front of the Church  
Fig 13 View of the back of the Church  
Fig 14 View of the crypt

Behind the main façade the Church presents a second line of wall that supports a choir accessible from two symmetrical spiral staircases in the corners as shown in Fig 13.

At 13 m a coffering ceiling closes the Church supported by a 6 iron trusses spaced 6m each other and connected in the longitudinal direction. A total of 32 windows of the same dimensions can be counted in the Church area, of which 8 are located in the main facade. Here the space is dominated by the central entrance with its white frame and by the other two lateral doors. Two small windows in the pediment illuminate the roof space.

Under the Church there is an underground space accessible from the connection spaces under the tower that houses a peculiar crypt with square plan 9x10.5 m covered by a concrete arch with symmetric holes housing windows obtained in the floor of the Church.

The distribution of the underground space starts with the crypt and, passing throughout the tower, continues in the east part of the building where it houses several rooms dedicated to administrative functions, stocks and library. From the entrance in the east area a stair connects the 3 administrative floors (one of which is in the underground). The east façade is characterized by 3 rows with 5 columns of openings for the two upper levels and for the underground floor. In the main floor the central opening is the back access to the Church whereas from the upper floor it is possible to access the open space of the tower.

The corridors under the tower have then the function of dividing the religious part from the other spaces. The remaining part of the volume of the tower is characterized by an open space with a ramp that follows the perimeter of the walls up to the bells area.
The tower is placed between two pyramidal elements up to half of its height. In this way it has the same width of the building around it. The volume inside measures around 20x5 m in plan and up to an height of around 30 m it is an open space with a ramp following the perimeter of the walls up to the bells location. On the top a room houses the three bells and allows the access to the exterior of the roof. The final mark of the tower is a decorative element that defines the end of the volume of the building: a spherical shape with a cross on the top and with the three golden painted letters “I” “H” “S”, an abbreviation for Jesus. However, a more detailed analysis of the geometry of the tower will be given in the chapter 4 as it will be the subject of a more detailed structural analysis.

The symmetry of the structural elements is maintained in the openings that are located on the top of the Church and of the administrative area and that surround the volume going all around it following a straight line and a rhythmic distribution.

The tower is characterize by two big circular openings that will be described more in detail in the chapter 4. One of them, the one facing the Church, houses a very peculiar clock. As explained in the previous chapter, the good relationship between the architect and the new President Masaryk apparently was the inspiration for this big opening facing directly the castle and the Cathedral on the other side of the river.

3.2 Technology identification

The technology identification of the building has to identify the composition, connections, construction techniques and arrangement of all the elements of the building. In order to be sure that all the elements of the building would have been described the classification used by Rohrich ([8],[9]) was followed. The building elements were subdivided into seven groups:
Analysis of the faults in the Church of the Sacred Heart in Prague

- foundations;
- vertical elements;
- horizontal elements;
- arches and vaults;
- roof covering;
- stairs;
- balconies.

The data available from the previous documentations ([2], [7], [3]) including the testing results were taken into account and verify, when possible, during the surveys. In general, the data collected during this study agreed with the literature and add more detailed information in some of the inspected areas. A resume of the main results of the testing phase performed during the 2011 in the occasion of the opening of a parking in the east side of the Jiřího z Poděbrad square is shown in the Annex 2

3.2.1 Foundation

After discovering the scarce capacity of the soil the project was changed and reinforced concrete foundations were designed. A concrete slab foundation was built under the underground floor from the crypt to the administrative area.

The pictures below belongs to an historical photographic report of the building works [3]. In particular, Fig 20 shows the excavation for the creation of the reinforce concrete slab under the tower and the east area. From cores extracted in the previous survey [3] the presence of a superficial floor layer of 30 cm was detected over the foundation slab which thickness is around 1.2m.

![Construction site at the beginning of works 19/8/1929](image1)

![Excavation for the reinforce concrete slab](image2)

In order to arrive to the strong ground deep strips of concrete were built as foundation for the peripheral walls of the Church with a thickness of 1.35m [3]. The depth of this foundation is around 6 m and the most
probable vertical section is shown in the image below. Moreover during the testing phase performed in the latest past survey [3] it was no possible to characterize the reinforcement of the concrete with a rebar detector up to a depth of 12 cm.

On the main façade a special solution was performed. In the west side of the Church, in fact, the wall of the façade is assisted by another wall needed for supporting the choir. The solution in the foundation was the production of an unique reinforced concrete foundation that follows the shape of the three entrance box. In Fig 25 and Fig 24 is shown the relevant hole excavated with a depth of 6m.
3.2.2 Vertical elements

Different morphologies of walls can be identified in the building. The distinction is not only between the vertical elements underground and the upper ones but also between different areas into the same level of the building. The characteristics were analyzed throughout testing in the previous survey [3] and the main results and a briefly description of the experiments needed will be reported in this work. In particular NDT and DT were preformed in several points of the building as shown in the Annex 2. The location of the surface hardness tests and the location of the cores extracted were matching so that it was possible to correlate the results in order to define the properties of the materials as will be shown in the chapter 3.4.

Concrete structure composes the walls in the underground floor and sometimes concrete is furbished with masonry of full bricks. This was discovered in the last investigation available in literature [3] contrary to the assertions of the previous documentation [7]. In particular this double wall was built at the base of the tower area as shown in the Annex 2.

Masonry walls can be then identify in the remain part of the structure. The tests results turn out that the composition of the masonry leafs is different in the height and between different parts of the building.

The peripheral walls of the nave consist on a multileaf structure produced by cement mortar joints with 3 types of bricks:

- dark orange perforated face bricks are used on the interior side (TYPE 1);
- the core consists of full concrete bricks (TYPE 2);
- dark brown perforated face clinker bricks are used on the exterior (TYPE 3).

In order to discover it during the survey conducted by [3] two holes (65 mm of diameter) were drilled in the north wall as shown in the Annex 2: one passing through hole and the other one for a thickness of just 30 cm.
During the construction of the walls lighter colour concrete blocks were embedded into the masonry in a rhythmical configuration, giving the impression of ermine coat, symbol of royal Highness.

In the upper part of masonry, where a plaster layer cover the bricks, it was presumed that the morphology of the wall consists in just full bricks.

The thickness of the three layers of the masonry was evaluated considering the dimensions of the cores extracted. The external layer seems to be 20 cm deep, followed by the massive leaf of 0.65 cm of full bricks. This internal part of the masonry is covered in the interior by an hole bricks layer of 20cm.

The composition of the layers of the tower’s walls is different from the one of the Church’s walls and in particular two layers can be identified:

- full bricks on the interior (TYPE 4);
- dark brown salt-glazed perforated clinker bricks on the outer side (TYPE 3).

Also in this case, where there is a render on the exterior side, it was assumed in the literature that the masonry is just made of full bricks.
At the height of around 20 m two big openings were built with an iron frame with a cross shape and a concrete frame around it as will be explained in detail in chapter 4.

### 3.2.3 Horizontal elements

It was possible to identify five different typologies of floors in the building.

The main façade access is elevated by an entrance floor that presents a bituminous layer on the top made of a light screed. An old drainage system is present on the edge wall at 60 cm from the ground level.
The Church is characterized by a typical terrazzo floor of grey colour with decorations regularly repeated in all the space: red and black circular shape elements are alternated as can be seen in Fig 12 and Fig 35.

On the edge area in contact with the walls the floor turns in square whitish mosaic around 90 cm wide (60 cm when the wall is thicker) as shown in Fig 36.

From the destructive test done in the pavement inside the Church (as shown in Annex 2) – performed removing one of the black circular decorative element of the floor in such a way to bring the minimum damage- it was possible to identify the thickness of the different layers of the floor.

As shown in the figure below under the terrazzo floor there is a concrete screed of around 65 mm. On the bottom a concrete layer 125 mm divides the structure from the soil.

Concrete floor characterizes the area under tower and the connection volume in the administrative area between the different rooms.

It was not possible to inspect the different rooms in the east part of the building during the inspection days but considering the old drawing available [7] these areas seems they seem to be characterized by timber structures with a screed above; his hypothesis should be proved with testing.

In the tower the floor consist in a ramp that goes up to the belfry closed with a floor with a hole in the middle. The description of the detail of this element will be given in chapter 4.
3.2.4 Arches and vaults

The structure doesn’t present curved elements covering the different spaces apart from the crypt that is closed with a vaulted structure. As can be seen in the pictures below, it appears that an external layer of masonry was realized as retaining wall. After that an arch shape scaffolding was prepared for covering the space with concrete taking care of the windows opening later added to the project.

![Fig 38 Lateral walls completed with the scaffolding; crypt vault](image)

![Fig 39 Formwork of the crypt](image)

3.2.5 Roof covering

The roof structure of the administrative area and of the Church is different compared to the one of the tower. A visual inspection was possible in the roofs of the Church and of the tower whereas it was not possible to access and detect the administrative covering structure. However from the old historical plans [7] this part seems to have the same composition of the Church one.

The roof covering the nave consists of several wrought iron trusses, spaced 6 m, over which timber purling with planking are placed. The height is 3.8 m in the central point and around 2.3 m at the wall level.

![Fig 40 Section of the roof trusses](image)

The iron structure supports a timber coffer ceiling that cover all the Church area. It is composed by squared shapes rhythmically spread with smaller square shapes inside. A floor built with a mix of wood and concrete covers the top part of the coffer ceiling where circular holes geometrically spaced house the old illumination system still active in the Church.
At the end of the ramp on the top of the tower, through a staircase is it possible to access the bells area. The room is illuminated by the several windows and houses a steel structure anchored in the masonry used for supporting the three bells. The top of the tower is closed by a simple timber structure with a concrete beam in the middle. This has also to support the big circular decoration outside supported by a steel structure located inside the bowl, inserted in the concrete beam and connected finally to the two longitudinal masonry walls. Small section purlings support a timber board and a final copper layer as in the roof of the Church.
3.2.6 Stairs

There are several vertical connections inside the building shown in the plan below. The main stair is in the middle of the administrative area and connects the several floors permitting the access to the open space of the tower. In order then to arrive to the belfry the connection is a cantilevered ramp following the perimeter of the main walls. A door divides the open space from the top room accessible through a staircase. Here it is possible to exit on the roof of the tower using back stairs.

The Church is accessible from three external staircases: the rounded central one and two symmetric side stairs. Moreover in the interior space it is possible to find two important connections to the choir and to the crypt. The access to the balcony in the back part of the main façade is throughout two spiral staircases located in the corners whereas the crypt is accessible from a staircase located behind the main altar or from the stair in the administrative area passing through the corridor under the tower.

3.2.7 Balconies

The most important balcony presents in the building is definitively the one in the back of the main façade housing the choir with the majestic organ. The railing with its succession of rounded white elements comes out in the reddish masonry of the Church giving the impression of a real balcony.
The same kind of rounded elements can be found in the two balconies at the base of the pyramids on the sides of the tower. One of them is accessible from the tower itself and represents the only access to the exterior of the roof of the Church.

![Fig 51 View of the detail of the railing](image1)

![Fig 52 View of the left balcony under the pyramid](image2)

### 3.3 Materials characterization

It was possible to obtain some material characterization from the experiments performed in the previous literature [3]. In particular, the cores extracted were tested in compression and other non-destructive tests (Hardness Tests) were performed in several location on the concrete, on the bricks and on the mortar. The compressive strengths were calculated referring to Czech norms and always in according to the Eurocodes.

#### 3.3.1 Concrete

The concrete is present in different parts of the structure: (a) foundations slab, (b) strip foundation of the Church, (c) massive foundation of the west façade, (d) walls of the underground floor, (e) ramp of the tower, (f) frame of the openings including the two big openings in the tower(g), (h) some of the floor of the administrative part and (i) on the top of the masonry for unifying the structure.

Destructive tests were applied together with non-destructive tests as the Schmidt Hammer test and the Rebar Detector (able to localize reinforcement up to 120 mm) as shown in the Annex 2. The cores extracted were tested in compression in order to classify the concrete.

The results obtained show a concrete C12/15 in the foundations (a, b, c). The same class of concrete characterizes the underground walls (d), whereas the ramp and the frame of the openings were not tested.

It is important to say that no reinforcement was found in the core drilled in (a). Moreover, after the removal of the sample it was possible to detect a fine grained clay soil. No tests were performed in the concrete of the tower that was assumed for further analysis of the same composition of the one detected in the other parts of the building.
Finally, in the Church the extracted core in the terrazzo floor revealed a very porous concrete. After a compressive test performed on the sample it was possible to characterize the concrete as C 8/10.

### 3.3.2 Bricks

In order to define the average compressive strength of the bricks destructive and non destructive tests were performed during the previous survey [3] with the results below. As explained in chapter 3.2.2 four types of bricks were detected in the structure:

- TYPE 1: dark orange perforated face bricks, used in the interior of the Church;
- TYPE 2: full concrete bricks, used as core layer of the Church masonry;
- TYPE 3: dark brown perforated face clinker bricks, used as exterior layer of the Church and of the tower;
- TYPE 4: full bricks, used in the interior layer of the tower and in all the thickness of the wall when it is plastered – top of the Church, administrative area and of the tower.

In order to find out the mechanical properties different procedures were performed. For the TYPE 3 a piece of a brick founded during the pit test F3 (see Annex 2) was tested, whereas for the TYPE 2 part of the core 1 extracted in the Church side wall was tested. Schmidt Hammer test was performed in all the locations: in the TYPE 2 and TYPE 3 the value obtained was correlated with the destructive tests performed. This NDT was even more important for TYPE 1 and TYPE 4 were destructive tests weren’t performed and a more conservative procedure was followed in this case. The average compressive strength of each type of brick is shown in the table below.

<table>
<thead>
<tr>
<th>Bricks</th>
<th>Average compressive strength of bricks determined by tests $f_c$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>TYPE 1</td>
<td>20.5</td>
</tr>
<tr>
<td>TYPE 2</td>
<td>20.0</td>
</tr>
<tr>
<td>TYPE 3</td>
<td>33.3</td>
</tr>
<tr>
<td>TYPE 4</td>
<td>19.0</td>
</tr>
</tbody>
</table>

Table 2 Average compressive strength $f_c$ of bricks determined by tests [3]

### 3.3.3 Mortar

The results obtained from the non destructive tests- i.e. the number of hits that an hammer (1 kg) performs from a distance of 0.2 m up to a depth of 5 mm – can be correlated with the compressive strength of the mortar. The compressive strength of mortar $f_{m}$ obtained is 1 MPa. However it must be considered that this test, suitable for lime and lime-cement mortar with $f_m > 10$ MPa, has an high level of uncertainty around 20%.
3.3.4 Masonry

The masonry strength was defined considering the results of the destructive and non-destructive tests performed on mortar and bricks as requested in accordance with the Eurocode 6. As four kinds of bricks can be identified, different compressive strength values were carried out for each typology. From these values a general compressive strength of each kind of wall was defined.

Based on these values and on the composition of each wall characteristic and recommended design values of masonry compressive strength of the individual structures of the building were carried out. Finally, as for the many types of masonry present the compressive strengths turned out to be very similar; so the values obtained were simplified and rounded.

<table>
<thead>
<tr>
<th>Type of bricks</th>
<th>Masonry strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Characteristic compressive strength ($f_{ck}$)</td>
</tr>
<tr>
<td>TYPE 4</td>
<td>$3.15$</td>
</tr>
<tr>
<td>TYPE 1</td>
<td>$3.25$</td>
</tr>
<tr>
<td>TYPE 3</td>
<td>$4.45$</td>
</tr>
<tr>
<td>TYPE 2</td>
<td>$3.20$</td>
</tr>
</tbody>
</table>

Table 3 Characteristic and recommended design strength of identified types of masonry

<table>
<thead>
<tr>
<th>Walls</th>
<th>Masonry strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>characteristic $f_{ck}$</td>
</tr>
<tr>
<td>A Peripheral walls of the nave (except upper parts covered with plaster)  - equal contribution of the different bricks was considered</td>
<td>$3.63$</td>
</tr>
<tr>
<td>B Tower – 1st and upper floor  - only full bricks. Ratio of interior perforated bricks in the wall adjacent to the nave was neglected, considering the very similar compressive strengths of bricks</td>
<td>$3.15$</td>
</tr>
<tr>
<td>C Tower above the nave (without belfry)  - with wall thickness 1050mm, considering 1/3 of exterior perforated bricks and 2/3 of full bricks</td>
<td>$3.58$</td>
</tr>
<tr>
<td>D Tower – belfry  - only full brick masonry was considered</td>
<td>$3.15$</td>
</tr>
<tr>
<td>E Other masonry structures  - only full brick masonry was considered</td>
<td>$3.15$</td>
</tr>
</tbody>
</table>

Table 4 Characteristic and recommended design values of masonry compressive strength of the individual structure of the building

<table>
<thead>
<tr>
<th>Structure</th>
<th>Design strength of masonry $f_{cd}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>with dark brown perforated facing bricks on the exterior side</td>
<td>$1.35$</td>
</tr>
<tr>
<td>other masonry structures</td>
<td>$1.30$</td>
</tr>
</tbody>
</table>

Table 5 Design values of masonry strength of the building
3.3.5 Stone

Stone elements are present in the building as non structural components. They can be detected mainly with two functions:

- Baseboard: this element, with a thickness of 25-30 mm and an height of around 50 cm surrounds the building base;
- Decorations: the statues of the exterior of the Church were mainly built with stone artworks.

3.3.6 Metals

Several types of metals can be identified in the structure. They can be structural elements, i.e. the bells supporting steel structure, or non structural including also the simple decorative function.

The metal usage in the building can be reported as shown below:

- copper used for the last level of the roof;
- wrought iron for the trusses of the roof structure of the Church;
- wrought iron for the structure supporting the bells;
- iron profiles for the frame of the big opening in the tower;
- steel profile for the ramp;
- steel elements inserted in the decorative concrete elements;
- reinforcement of the concrete, if present.

3.4 Damage identification

The building does not exhibit any visible severe structural crack pattern, which could affect the stability and safety of the structure. This was also proved in the last survey report [3] where the static structural assessment of the Church was studied in order to build a parking in the west area of the square where the Church is located. The analysis was performed according to Eurocodes and a safety factor of around 2 was carried out.

The main damage patterns that can affect the structural behaviour of part of the building were identified in the tower and will be explained in detail in chapter 4. In particular a first group of cracks affects the longitudinal walls of the tower around the two big circular openings and a second crack pattern can be identify along the ramp.

However, in general, several non structural damage were detected in the exterior and interior parts of the building during this work and reported in detail in the following annexes:

- Annex 4 - Photographical survey and damage identification of the main façade (west)
Analysis of the faults in the Church of the Sacred Heart in Prague

- Annex 5 - Photographical survey and damage identification of the south façade
- Annex 6 - Photographical survey and damage identification of the north façade
- Annex 7 - Photographical survey and damage identification of the east façade
- Annex 8 - Photographical survey and damage identification of the interior of the Church

Moreover, a classification of the damage identified was carried out considering the ICOMOS glossary as shown in the Annex 9 - Table of damage identification (ICOMOS glossary).

An important crack pattern identified during the damage analysis is located in the concrete wall supporting the entrance floor. This phenomena is considered a local damage, not affecting the structural stability of the overall building. However, considering that the evolution of this damage can cause safety problems for the people accessing the Church, the rehabilitation of this part is recommended, also considering the aesthetical reasons.

The wall, as shown in the pictures below, presents several cracks rhythmically spread in its vertical surface following the location of the drainage system pipes. Another problem linked with water is due to the lack of a proper slope in the entrance floor that allow the presence of several puddles.

![Fig 53 Accumulation of water in the floor](image1)
![Fig 54 Several cracks in the front wall related to the drainage system of this area](image2)
![Fig 55 Detail that shows the severity of these cracks](image3)

Fig 53 Accumulation of water in the floor
Fig 54 Several cracks in the front wall related to the drainage system of this area
Fig 55 Detail that shows the severity of these cracks

Others cracks were detected on the top of the pediment on the east and west façades. These can probably be associated to local phenomena, mainly due to the connection between different materials.

![Fig 56 View of the east façade: identification of the crack pattern in the top of the pediment](image4)
![Fig 57 Detail of the west façade: crack on the top of the pediment under the central statue](image5)

Fig 56 View of the east façade: identification of the crack pattern in the top of the pediment
Fig 57 Detail of the west façade: crack on the top of the pediment under the central statue
The dominant cause of the numerous micro hair cracks in the masonry is the volume change of different structural materials due to long-term effects of thermal cycles, together with the action of water. This widespread phenomena appears mostly in the bricks near the concrete decorative rectangles and in the mortar joints where in the location here high stresses appears some of the material is lost.

Also the baseboard stone suffer this differential expansion of the material and it completely crush in some part of the building as shown in Fig 60.

The main problematic agent is the water affecting the building causing moisture problems, erosion of the stones and sustaining a favourable environment for the formation of black crust and efflorescence. Also some damage on the drainpipe cause local deterioration.

Moisture principally attacks the white plaster on the outer and inner side of the walls causing its detachment from the wall (blistering) and its loss. The leaking of water and the presence of capillarity rise affect also the baseboard stones that present erosion phenomena.

The elements most exposed to the aggressive environment present black crusts and in some places as shown in Fig 63 also efflorescence due to the evaporation of saline water in the porous stones.
Also the metal elements present in the exterior part of the building, as doors, railing, final copper layer of the roof, show the typical discolouration and presence of rust, due also to a lack of maintenance during the years. In the main façade some metal bars are missing and its presence in the past is visible in the anchorages points to the railing frames.

Also inside the Church the presence of salt can be detected in the bottom part of the masonry, where the mortar joints show a whitish colour.

The main non structural problem present in the interior can be also in this case attributed to the different thermal behaviour of near materials: the terrazzo floor shows an important crack pattern continuing also in the near mosaic. In some areas there is the presence of a previous “rehabilitation” with the application of mortar for covering the faults.

Another problem affecting the safety of the believer is the old illumination system embedded in the coffering ceiling. Several times in the past it experienced the loss of some of its components [2] luckily without causing any other damage.
Moreover in the connection between the main façade wall with the roof the timber beams are directly connected with the masonry and the relative movements between these elements together with the presence of moisture causes a diffuse damage among all the connection area.

Fig 72 Connection between the main façade wall and the roof
Analysis of the faults in the Church of the Sacred Heart in Prague

Erasmus Mundus Programme

ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS
4. VISUAL INSPECTION AND DAMAGE IDENTIFICATION OF THE TOWER

The great volume of the tower is inserted in the building and divides the administrative area from the Church. Its rectangular area is characterized by a corridor in the underground and main floor with the side entrances to the building. Continuing upward the most significant part of the tower consists in an open empty space with a ramp that allows the access to the belfry which represents the top of the tower. The space is illuminated by two big circular openings that are the key points of this structure. The west opening houses also the clock of the Church. Other openings are located in the two short sides but their extremely small dimensions makes their contribution to the interior illumination secondary. On the top of the tower the bell’s room presents 14 openings, without glass, perfectly symmetric.

Fig 73 View of the tower. From left: west side, east side, south side (x2), north side

The rectangular open space area is 4.6 m width and covers the same length of the Church around 27 m even if the two lateral edges (around 2.9 m long) can be considered as a separate part as they are constrained by walls in the underground and main floor, defining respectively a basement window and the entrance. Continuing upward these lateral volumes detach from the real space of the tower shaping two bricks pyramids in the north and south sides.

Fig 74 Views of the tower volume

The thickness of the walls changes going upward and within the same level. In order to evaluate the behaviour of the tower in the next steps of the work this geometrical feature had to be taken into account subdividing the space in several levels with constant thickness as shown in the Annex 11 - Characterization of the structure of the tower. Up to the level of the roof of the main building the thickness of the long wall is
around 1.8 m. Then it decreases to 1.2 m and it remains the same up to the belfry in which the thickness is just 0.9m. The short walls are more regular and their 1.05 m of thickness will remain the same up to the bells area where it will decrease to 0.85 m. These measure were carried out from the old plans [7] and through measurements carried out in situ during the inspection days. The composition of the walls in the open space of the tower, as shown in chapter 3, consists in an external layer of salt glazed clinker bricks that was assumed 20 cm thick from the core extracted in the main nave wall [3]. Then, in the interior, a white plaster covers a layer of full bricks masonry with variable thickness.

The façades of the tower are affected by some non structural damage mostly concentrated in the most exposed areas and characterized by the detachment of the white plaster. As shown in the Annex 10 - Photographical survey and damage identification of the exterior of the tower, the pediment and the lintel of the west side are affected by several cracks presumably related to the differential thermal behaviour of adjacent materials. Furthermore, the two small terraces at the bottom of the pyramids presents concrete rounded elements suffering the expansion of the iron inside due to its corrosion.

The most significant damage connected to the structural behaviour of the building is detectable in the concrete ring frame of the circular openings. During the surveys it was possible to analyze the crack pattern around the west opening whereas it was not possible to access the roof of the administrative east area and an old damage plan [3] was considered in the Annex 10. Cracks develop in the radial direction of this circular element as shown in Fig 76 and in the Annex 3.

The same consideration done in the previous chapter can be taken into account for the external clinker bricks layer and concrete decorative elements. Also the external layers of the roof of the tower present the same composition of the ones of the Church and involve the same damage.

The main relevant problems that can affect the structural safety of the tower can be identified in the ramp and in the longitudinal walls.
The two longitudinal walls are mainly characterized by the presence of two glassed circular openings. The openings with diameter of 7.6 m are circumscribed by concrete rings 1.3 m wide and 0.6 m thick. The structure that supports the glassing of the openings consists in a principal frame and two substructures. The principal frame consists of two iron cross beams and an iron ring beam. The vertical and ring beam have U-shape cross-section, while the horizontal beam has a box cross-section as shown in the Annex 14 – Clock opening structure. The ring beam is fixed to the concrete around with bolts identified during the visual inspection. Moreover an iron corroded element connecting the iron ring with the concrete one was found in the west wall (figure O11 of the Annex 14): the corrosion of this element and its subsequent expansion contributed to the crushing of masonry in this area. A deeper analysis of the connections between the iron frame and the concrete beam is however recommended in order to detect the presence and the conditions of other possible steel elements inserted in the concrete.

The other two substructures don’t have any structural function but they compose the grid in which the glass panes, with a fix dimension of around 51x48.5cm, are framed.

The main crack pattern that affects the walls is present around the opening in the connection between the concrete element and the masonry and in the concrete element itself as shown in the Annex 13 - Photographic survey and damage identification of the walls of the tower. Mainly two classes of cracks can be defined: one develops in the radial direction from the iron frame into the concrete and sometimes it continues in the masonry around; other cracks follow the connection between concrete and masonry. Loss of material phenomena are detectable in the connection between the iron and the concrete in the west façade (Fig 79). In this area it is possible to see an iron bar completely rusted probably used as a connector.
Analysis of the faults in the Church of the Sacred Heart in Prague

Fig 79 Views of the damage pattern detected in the west opening. A more detailed photographic survey is available in the Annex 13.

Fig 80 Crack pattern around the openings on the interior side of the west and east walls

On the west wall it was possible to survey the outer and inner side of the wall. Due to that it was detected the presence of cracks on the concrete in the same location of the one visible in the inside.

The main volume of the tower is accessible from the upper floor of the administrative area. This open space is dominated by a white plastered ramp that follows the perimeter of the walls and leads to the bells area in the top.

This ramp is cantilevered in the walls except where the openings appears: in these areas the ramp is detached from the main walls (Fig 83) and it is supported only at the ends, where it connects to the cantilevered parts. This was called “suspended” ramp. The railing is usually a concrete element uniformly connected with the ramp itself. In R6, R7, R8 where the ramp is not directly cantilevered the railing consists in three horizontal metal elements connected vertically at regular intervals.
The composition of the ramp is still unknown as no previous documentation is available – in the previous surveys and in the old plans available it was supposed to be a concrete slab structure with 18-20 cm of thickness- and no tests were performed in the short time of this work. From the visual inspection the cantilevered ramp seems to be a reinforced concrete structure monolithic together with its railing whereas further considerations should be said about the “suspended” parts.

In fact, from a damaged area in the edge of this segment of ramp it was possible to detect a steel profile with apparently “I” shape as shown in the Fig 84 - picture P17 of the Annex 12 - Photographical survey and damage identification of the ramps. The structure between the two profiles should be investigated with NDT as GPR in order to detect the composition. It can be hypnotized that the composition of the ramp is characterized by two steel profiles connected with steel plates or just by concrete.

The second significant crack pattern detected during the visual inspection affects the ramp and its railing. This damage is periodically repeats: the cracks develop in the perpendicular direction to the development of the ramp itself and continue up to the top of the railing. Although a deeper study of the element would be
recommended in order to understand the composition and the general structural behaviour this crack pattern seems to be caused by shrinkage of the concrete in an early moment.

On the other hand, ramps R6, R7, and R8 are affected by a more severe damage that is mainly concentrated in the not cantilevered part. Cracks develops at the top and at the bottom of the two edges and proceed up to the railing causing loss of material in the connection between concrete and metal (Fig 86).

Also the presence of non structural damage due to the moisture was detected. The most evident phenomena was identified in the R9 (Fig 85). This phenomena usually results in detachment of plaster, discoloration and in some more exposed areas in the desegregation of the plaster itself.

It is possible to suppose that the two phenomena explained above can be related each other or that they connect in the critical points in which the ramp is divided from the walls near the circular opening. Here the cracks in the wall and the ones in the ramp are laying in the same line. An analysis in order to understand the relation between this two phenomena is strongly recommended after a deeper evaluation of the composition of the ramp itself.

The next step in the evaluation of this case study has so to be divided in the evaluation of the causes of these two structural damage identified. Primarily importance was given in this thesis to the analysis of the thermal-mechanical behaviour of the masonry wall considering the peculiar opening size and different materials properties. On the other hand the safety of the ramp should be studied after discovering its morphology. The characteristics of the ramp should be investigated in the cantilevered part as well as in the “suspended” one. In particular, GPR tests are recommended. From the results of this NDT test in fact it is possible to define the material characterizing the slab, its thickness and the presence of steel bars inside. In the suspended ramps it is possible to find out the composition of the part in between the two steel profiles. Even hardness surface tests can be performed on the concrete surface in order to discover if it can be classified together with the one in the foundations and in the walls in the underground. The same test is recommended also for the concrete beam around the circular openings.

However, this work will focus on the study of the damage in the walls of the tower as it was not possible to develop the testing phase and the related analysis of the ramp during the thesis time. In particular, the analysis will focus on the west wall as it shows a more severe damage pattern compared to the one on the east side.
5. PRELIMINARY STRUCTURAL ANALYSIS OF THE TOWER WALLS

After the evaluation of the damage identification the study focused on the analysis of the structural damage affecting the wall of the tower. Previous studies ([2]) have attributed this damage to volume changes associated with temperature variations. However this hypothesis was not supported by a systematic analysis. Any intervention or restoration action should be preceded by a rigorous identification of the cause of existing damage. When thermal phenomena are suspected, the identification often involves a numerical thermo-mechanical analysis of the affected structure.

In particular the aim of this study will be the investigation of the thermal behaviour of the west wall in order to verify if changes of temperatures can be the cause of the detected crack pattern on the west wall, as shown in the previous chapter.

Through the diagnosis we will try to identify the causes of damage and decay and to characterize the present condition of the structure. Of course, the information obtained by this tool should not be in contradiction with that provided by the historical research and visual inspection.

The procedure with which the structural analysis was approached started with the definition of a simple initial 2D model. The purpose of this preliminary analysis was to verify if thermal load in a plausible range is able to induce in the structure such stresses, that might result in its cracking. Then, this model was modified and redefined in order to make it more realistic and more representative of the real structure obtaining better results.

A plane stresses analysis was performed with the ADINA software on the model considering both the changes of temperature applied to the overall structure and the temperature variation in some of its parts.

The geometry of the 2D model represents one of the longitudinal walls of the tower in between the two floors that delimit the open space. The area affected by the crack pattern is focused around the opening, whereas the other part of the masonry doesn’t show any significant structural damage.

All the vertical span of the masonry wall in between the two floors was meshed with 2D-solid elements with a thickness of 1.8 m at the bottom and 1.2 m in the area around the clock. The lower thicker part was included in the model to minimize the effect of the assumed boundary conditions on the stress state in the area of interest around the openings. Moreover in this way it was possible, checking the magnitude of the
displacements, to evaluate the possibility of neglecting this part of the wall in more complicated models considering it stiff enough in the vertical direction and replacing it with simple supports.

The concrete ring was represented with 2D-solid elements with a thickness of 0.6 m whereas the iron profiles of the principal frame of the window were represented by 2-node Hermitian beam elements. The horizontal beam was meshed with a box cross-section whereas the vertical beam and the ring frame with U-shape cross sections as shown in Fig 87.

The connection between the concrete ring and the iron frame profiles was modelled at this stage by imposing rigid links connecting the displacements of all the nodes in order to represent a fix boundary between the two elements.

The following kinematic boundary conditions were considered: displacement the bottom of the wall was fixed in vertical direction and the central point was fixed also in the horizontal direction.

Fig 88 Geometry, boundary conditions and pressure applied on the top of the linear model

In order to represent the real construction steps and relative development of stresses in the elements the iron elements will be added in the model after the application of the self weight of the wall using the “element birth” feature of Adina program.

In addition to the self weight a pressure equal to 0.02 MPa was applied on the top of the masonry for representing the cut part of the wall of the belfry.

In the first stage, all the materials were modelled as linear elastic, with parameters listed in Table 6.

The first load cases were studied considering elastic properties of the materials. A coefficient of expansion was estimated for each material as shown in the table below.

<table>
<thead>
<tr>
<th>Materials</th>
<th>E (Initial Young modulus, MPa)</th>
<th>v</th>
<th>Density (kg/mm³)</th>
<th>α (coefficient of thermal expansion)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 IRON</td>
<td>2.0 × 10⁵</td>
<td>0.3</td>
<td>7.75 × 10⁴</td>
<td>1.30 × 10⁻⁵</td>
</tr>
<tr>
<td>2 CONCRETE</td>
<td>2.5 × 10⁵</td>
<td>0.2</td>
<td>2.40 × 10⁴</td>
<td>1.00 × 10⁻⁵</td>
</tr>
<tr>
<td>3 MASONRY</td>
<td>1.0 × 10⁴</td>
<td>0.2</td>
<td>2.00 × 10⁶</td>
<td>5.00 × 10⁻⁶</td>
</tr>
</tbody>
</table>

Table 6 Characteristics of materials specified in the model
Loading was applied in the following steps (see Fig 89): first, the structure was loaded by self-weight since time 0. The self-weight included mass-proportional load due to gravity acceleration of 9.81 m/s² applied to all elements and the load from the upper structure not included in the model (belfry). The latter was represented by uniformly distributed pressure of 0.02 MPa acting on the top of the modelled wall (see Fig 88).

Secondly, at time 1 the elements representing the iron frame of the window were introduced using the “element birth” function. Mass proportional gravity load was introduced to these elements.

Thirdly, a thermal load was introduced at time 2. This was done considering several possible scenarios:

a) An uniform increase of temperature is applied equally in all the structure except the bottom thicker part of the wall;

b) A uniform decrease of temperature is applied equally in all the structure except the bottom thicker part of the wall;

c) The hypothesis that the iron elements warm up faster than the wall was studied. It was supposed that these elements warm up at the double rate compared to the other parts of the structure as shown in Fig 90;

d) The hypothesis that the iron elements cool down faster than the wall was studied. It was supposed that these elements cool down at the double rate compared to the other part of the structure as shown in Fig 90;
e) The effect of the warming up of the iron was considered applying a linear increase of temperature on the frame element and considering the other part of the structure at the same initial temperature (Fig 89);

f) The effect of the cooling down of the iron was considered applying a linear increase of temperature on the frame element and considering the other part of the structure at the same initial temperature (Fig 89).

In all the cases the temperature was varied in the range of +/-20°C from the initial state apart when a faster warming up of the iron element was evaluated. In that case the temperature of the iron increase or decrease up to +/-40°C.

The results in terms of principal tensile stresses band plot and vectors are shown in Fig 91 using no smoothing techniques and considering the result at the integration points.
The tensile capacities of the materials were evaluated 1/10 of the value of the compressive strength obtained from the testing phase of the previous surveys as described in the chapter 3.

<table>
<thead>
<tr>
<th>Tensile capacity (MPa)</th>
<th>Masonry</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.35</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Table 7 Tensile capacity of the materials

Fig 91 a) shows that when the structure is exposed to uniform increase of temperature by 20°C, the higher thermal expansivity and higher elastic stiffness of concrete result in build up of tensile stress of about 0.55 MPa in the masonry right above the concrete ring. Considering that the tensile strength of masonry was
assumed as 0.35 MPa, this stress should cause cracking. From the principal stress direction it can be concluded that the cracks should develop in the radial direction from the opening.

Fig 91 b) shows that when the structure is exposed to uniform decrease of temperature by 20°C, tensile stresses higher than 1.2 MPa, value assumed as tensile capacity of concrete, appear in the concrete ring. Higher concentration of stresses appears in the connection between the iron element and the concrete ring at the bottom of the opening. From the principal stress direction it can be concluded that the cracks should develop in the radial direction from the opening.

Fig 91 c) shows that when the iron elements warm up faster up to 40°C than the wall (up to 20°C), the higher thermal expansivity and higher elastic stiffness of concrete result in build up of tensile stress higher than the one obtained in the case (a) of about 0.6 MPa in the masonry right above the concrete ring. Considering that the tensile strength of masonry was assumed as 0.35 MPa, this stress should cause cracking. From the principal stress direction it can be concluded that the cracks should develop in the radial direction from the opening.

Fig 91 d) shows the case when the iron elements cool down faster up to -40°C than the wall (up to -20°C). All the concrete experiences a stress concentration higher than its capacity fixed at 1.2 MPa. The most stressed area is at the connection between the vertical iron element and the concrete ring at the bottom of the opening. In the area near the connection between the concrete ring and the horizontal iron profile the principal stresses direction is inclined defining a possible curved crack development.

Fig 91 e) shows that when the structure is exposed to an uniform increase of temperature by 20°C in the iron elements tensile stresses concentrated at the top and at the bottom of the opening. However the magnitude of these stresses is lower than the capacity of the materials. Then, the only warm up of the steel element shouldn't cause cracking phenomena.

Fig 91 f) shows that when the structure is exposed to an uniform decrease of temperature by 20°C in the iron elements tensile stresses arise up to a value of 1.6 MPa at the connection between the cross iron profiles and the concrete ring. Considering that the tensile strength of concrete was assumed as 1.2 MPa, this stress should cause cracking.

The initial calculations have shown that when the structure was exposed to thermal loading with highly simplified spatial distribution but within a reasonable temperature range, stresses exceeding tensile stress of the individual materials have developed in the cases (a), (b), (c), (d) and (f) whereas in the case (e) the only warm up of the iron profiles is not enough for reaching a level of stress higher than the capacity of the materials.

In order to represent in the most representative way the phenomena a refined model was considered. The refinement consisted namely in implementing a different geometry assumption regarding the connection between the iron profiles and the concrete beam was taken into account. Moreover, non linear characteristics of the materials were considered as shown in Table 8. These were implemented in the ADINA software using the concrete material model [10].
constitutive characteristics are such that the model can also be useful for representing masonry as a homogenised, quasi-brittle material. The model represents the material as homogenous isotropic, accounts for tensile failure using the smeared crack-band model, which is characterized by tensile strength, fracture energy and linear tension-softening relation. Compressive nonlinearity is treated by plasticity.

The post cracking tensile stress was not considered in the model. A fracture energy value \( G_f \) was estimated considering values used in literature according to the Model Code 90. The tensile strength of the materials was considered as 1/10 of the compressive strength.

The connection between the concrete ring and the iron frame profile was modelled with rigid links connecting the displacements of all the nodes in the cases in which the iron members expand and thus act on the concrete by pressure. When the structure iron element shrink just 8 rigid links were created in order to represent the fact that tension to the surrounding concrete is transferred only through the visible bolts or iron bars detected during the visual inspection (see Annex 14).

The same boundary conditions and loading scenarios as in the earlier discussed first-stage analyses were considered.

The results obtained are shown in Fig 93.

<table>
<thead>
<tr>
<th>Materials</th>
<th>( \frac{E_0}{\text{(initial Young modul}})</th>
<th>( v )</th>
<th>Density (kg/mm(^3))</th>
<th>( \alpha ) (coef. of thermal expansion)</th>
<th>( \sigma_t ) (Tensile capacity-MPa)</th>
<th>( G_f ) (Fracture energy)</th>
<th>( \sigma_c ) (Compr. capacity-MPa)</th>
<th>( \varepsilon_c ) (Max Compr. strain)</th>
<th>( \sigma_u ) (Ult. Compr. capacity-MPa)</th>
<th>( \varepsilon_u ) (Ult Compr. strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE</td>
<td>( \frac{2.5}{10^6} )</td>
<td>0.2</td>
<td>( 2.4 \times 10^{-6} )</td>
<td>( 1.0 \times 10^{-5} )</td>
<td>1.20</td>
<td>0.050</td>
<td>12.00</td>
<td>0.0020</td>
<td>7.0</td>
<td>0.0035</td>
</tr>
<tr>
<td>MASONRY</td>
<td>( \frac{1.0}{10^6} )</td>
<td>0.2</td>
<td>( 2.0 \times 10^{-6} )</td>
<td>( 5.0 \times 10^{-5} )</td>
<td>0.35</td>
<td>0.012</td>
<td>3.50</td>
<td>0.0020</td>
<td>2.5</td>
<td>0.0035</td>
</tr>
</tbody>
</table>

Table 8 Definition of the non linear parameters for the concrete and masonry
Analysis of the faults in the Church of the Sacred Heart in Prague
Fig 93 Results of the 2D non linear model
Fig 93 a) shows that when the structure is exposed to uniform increase of temperature by 20°C, the maximum capacity of the masonry and concrete is reached at the top of the opening. Due to that cracks appear in the masonry and on the concrete. From the crack plot it can be concluded that the cracks should develop in the radial direction in the masonry following mostly three lines vertically symmetric.

Fig 93 b) shows the case when the structure is exposed to uniform decrease of temperature by 20°C. The model predicts formation of cracks at the connections with the cross iron profiles as can be seen from the plot of the principal strain. Moreover this crack pattern continues in the masonry at the top and at the bottom of the concrete ring developing in radial direction.

Fig 93 c) shows that when the iron elements warm up faster up to 40°C than the wall (up to 20°C) the maximum tensile capacity of the concrete was reached by a larger area of the ring compared to the case (a) and develops also in the masonry in a radial direction.

Fig 93 d) shows the case when the iron elements cool down faster up to -40°C than the wall (up to -20°C). The model predicts formation of cracks at the connection between the concrete ring and the cross iron elements. Moreover this crack pattern develops also in the masonry at the top and at the bottom of the opening. The direction of the crack at the connection between the horizontal profile and the concrete ring is partly radial and partly tangential.

Fig 93 e) shows that the magnitude of tensile stresses reached when the structure is exposed to an uniform increase of temperature by 20°C the iron elements is not enough to exceed the tensile capacity of the materials and then no cracks appears.

Fig 93 f) shows that when the structure is exposed to an uniform decrease of temperature by 20°C in the iron elements the tensile capacity of concrete is reached in the connection between the horizontal iron element and the concrete ring causing possible cracking.

Based on the results obtained it was possible to conclude that a change of temperature in the wall can be the origin of the crack pattern in the masonry. For this reason a deeper analysis will be performed defining more in details the thermal loads and other model parameters.

When thermal phenomena are being evaluated it is necessary to be able to reproduce the real temperature history that affects the structural element. This is possible through the monitoring that is increasingly becoming a very helpful tool for evaluating the behavior of the structure and of the variables that act on it. So the first redefinition of the model had to be the identification of the real thermal load affecting the wall during the time. As these data were not available and in order to be representative the recorder should last at least one year a different approach for obtaining the monitoring results was applied as will be shown in the next chapter.
6. EVALUATION OF THE TEMPERATURES ON THE WALLS OF THE TOWER USING A BUILDING PERFORMANCE SIMULATION PROGRAM

The boundary conditions for the thermal simulation are usually determined by monitoring of temperatures and of others thermal parameters. However, for the analysis to provide reliable results, the monitoring should be performed at many spots and over a long period of time, which was not possible during the thesis time and in general it may not be feasible in many practical situations.

In this thesis was studied the possibility to replace the monitoring by employing the Building Performance Simulation (BPS), which is commonly used for the evaluation of building energy performance and indoor environmental quality, in order to obtain the internal and external surface temperature histories of structural elements (walls).

The BPS calculation makes it possible to take into account the climatic phenomena as solar irradiation, air temperature and humidity, wind velocity and direction as well as the indoor operation on the scale of the building (macroscale) over a long period of time (years) with fine time resolution (minutes). The heat transfer calculations in the individual structural elements, however, are based on a simplified one-dimensional modeling.

The results obtained will be then used as boundary conditions for more detailed thermal analysis on the scale of the structural elements (mesoscale). And then this thermal field is then utilized as loading in stress and fracture analysis.

6.1 DESIGNBUILDER software

6.1.1 Geometry of the model

In order to build the 3D model in the software the most reliable simplified geometry of the building was considered as shown in the Annex 11. In particular, assuming that the tower represents a volume almost geometrically and energetically independent from the other part of the building just the model of the tower was built with “building block” elements and the surrounding part was added as several “adiabatic component block” elements; the floor was modelled as a “ground component block” element. The program automatically sets the adjacency of the surfaces of the studied zone with the one of the components. In the case of the adiabatic surfaces it is considered that no heat transfer is present across the external surface. This typology of component was used for modelling the pyramidal volumes linked to the short sizes of the tower as well as for the Church, administrative area and crypt. All the tower volume from the underground floor up to the belfry was modelled.

The tower was subdivided in several zones in its height as the thickness of the masonry decreases going upward. At the end eight building zones were built with a constant thickness of walls each and all of them were automatically connected by the program in the same zone.
Three typologies of openings were realized in the walls. In the bell area the openings don’t have any glass, just an old timber shutter so they were modelled as a ventilation openings. There are six openings in each long side of the tower and one in the short sides, each with dimensions of 1.65x3 m. The circular openings were drawn in order to obtain the real dimension and location. Between the typologies of windows available in the program it was selected a single glass window with a very high transmittance around 6 W/m²K in order to represent the actual condition of the windows, of its connection with the concrete ring and of the glasses some of which are also broken. Finally, other two small openings of 1 m² each were built in the north and south sides of the tower.

![Fig 94 Views of the 3D model in the DesignBuilder software](image)

Some more consideration had to be done in order to be able to evaluate the exterior and interior surface temperature of the concrete ring around the opening. As it was not possible, due to limitations of the software, to model the concrete rings at their real position around the main windows, they were represented by two horizontal strips of concrete, 60 cm thick, one in the east and one in the west façade. The strips were located in the same zone as the windows. Considering that the software calculates thermal transport at the scale of zones, this geometrical simplification had negligible effect on the calculated temperatures.

### 6.1.1 Materials characterization of the model

As the walls have different thickness, in different levels and within the same level, several typologies of masonry were specified to the program as shown in the table below. The massive masonry that characterizes this historical buildings was performed in the program considering its several layers. Moreover each layer thicker than 50 cm had to be subdivided in more elements as the program has size limitation for the composition of the walls.
The most appropriate thermal characteristics input for the layers’ material would have been the ones evaluated from the direct testing of the materials that compose the structure. As no test was performed some general assumptions were taken into account and the properties of the selected materials are shown in the tables below. The internal plaster was considered as a lime/concrete material, the wall was considered continuous and the two materials composing the masonry were taken into account selecting different properties of the materials proposed by the program that could be representative of the thermal behaviour of the real ones. Also for the concrete it was supposed a low quality material assuming it to be of the same class C12/15 as the concrete tested in several different parts of the building (Annex 2).

<table>
<thead>
<tr>
<th>Area of the model</th>
<th>h</th>
<th>Wall</th>
<th>Tot. Thickness</th>
<th>Typology of masonry</th>
<th>Typology of floor</th>
<th>Typology of roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Underground)</td>
<td>5*</td>
<td>North</td>
<td>1.05</td>
<td>Masonry1</td>
<td>Concrete floor1</td>
<td>Concrete floor2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.8</td>
<td>Masonry3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>1.05</td>
<td>Masonry1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.8</td>
<td>Masonry2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 (Main floor)</td>
<td>5.2*</td>
<td>North</td>
<td>1.05</td>
<td>Masonry1</td>
<td>Concrete floor2</td>
<td>Concrete floor2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.8</td>
<td>Masonry4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>1.05</td>
<td>Masonry1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.8</td>
<td>Masonry4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 (Up to the back roof)</td>
<td>9.3*</td>
<td>North</td>
<td>1.05</td>
<td>Masonry1</td>
<td>Concrete floor2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.8</td>
<td>Masonry4</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>1.05</td>
<td>Masonry1</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.8</td>
<td>Masonry4</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>4 (Up to the Church roof)</td>
<td>3.16*</td>
<td>North</td>
<td>1.05</td>
<td>Masonry1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.35</td>
<td>Masonry5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>1.05</td>
<td>Masonry1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.8</td>
<td>Masonry4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5 (Under the clock)</td>
<td>3.18*</td>
<td>North</td>
<td>1.05</td>
<td>Masonry1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.2</td>
<td>Masonry6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>1.05</td>
<td>Masonry1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.2</td>
<td>Masonry6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6 (Under the clock)</td>
<td>Eq.</td>
<td>North</td>
<td>0.6</td>
<td>Concrete</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.2</td>
<td>Masonry6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>0.6</td>
<td>Concrete</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.2</td>
<td>Masonry6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7 (Clock area)</td>
<td>12.84*</td>
<td>North</td>
<td>1.05</td>
<td>Masonry7</td>
<td>-</td>
<td>Concrete floor3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>East</td>
<td>1.2</td>
<td>Masonry6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>South</td>
<td>1.05</td>
<td>Masonry7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>West</td>
<td>1.2</td>
<td>Masonry6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 (Bells area)</td>
<td>12.84*</td>
<td>Lower part of the roof=6.88*</td>
<td>North</td>
<td>0.85</td>
<td>Masonry9</td>
<td>Concrete floor 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Central part of the roof=8.36*</td>
<td>East</td>
<td>0.9</td>
<td>Masonry8</td>
<td>Timber Roof1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>South</td>
<td>0.85</td>
<td>Masonry9</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>West</td>
<td>0.9</td>
<td>Masony8</td>
<td></td>
</tr>
</tbody>
</table>

*the program include also the thickness of the upper floor

Table 9 Subdivision of the model in level with different masonry properties
<table>
<thead>
<tr>
<th>Masonry</th>
<th>Material Description</th>
<th>Thickness [m]</th>
<th>Conductivity (k) [W/(mK)]</th>
<th>Specific Heat [J/(kgK)]</th>
<th>Density [kg/m³]</th>
<th>Emissivity</th>
<th>Solar absorptance</th>
<th>Visible absorptance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry1</td>
<td>CEMENT/LIME PLASTER (int-tower)</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>1.05</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER (int-side pyramid)</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry2</td>
<td>CONCRETE WALL</td>
<td>1.65</td>
<td>2.3</td>
<td>1000</td>
<td>2300</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>0.15</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry3</td>
<td>CONCRETE WALL (Concrete reinforced with 1% steel)</td>
<td>1.05</td>
<td>2.3</td>
<td>1000</td>
<td>2300</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>0.75</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry4</td>
<td>CEMENT/LIME PLASTER (int- Church/adm area)</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>1.8</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
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</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
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<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry5</td>
<td>SALT GLAZED CLINKER BRICKS MASONARY (Burned Bricks)</td>
<td>0.2</td>
<td>0.85</td>
<td>840</td>
<td>1500</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
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<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>1.05</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry6</td>
<td>SALT GLAZED CLINKER BRICKS MASONARY</td>
<td>0.2</td>
<td>0.85</td>
<td>840</td>
<td>1500</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>1</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry7</td>
<td>SALT GLAZED CLINKER BRICKS MASONARY</td>
<td>0.2</td>
<td>0.85</td>
<td>840</td>
<td>1500</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>0.85</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry8</td>
<td>SALT GLAZED CLINKER BRICKS MASONARY</td>
<td>0.2</td>
<td>0.85</td>
<td>840</td>
<td>1500</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>0.7</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Masonry9</td>
<td>SALT GLAZED CLINKER BRICKS MASONARY</td>
<td>0.2</td>
<td>0.85</td>
<td>840</td>
<td>1500</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>FULL BRICKS MASONRY (WITH CEMENT MORTAR)</td>
<td>0.65</td>
<td>0.72</td>
<td>840</td>
<td>1920</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>CEMENT/LIME PLASTER</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Concrete</td>
<td>CONCRETE</td>
<td>0.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 10 Materials properties of the vertical elements
In order to perform this analysis on an historical building several considerations have to be done regarding the use and the occupancy of the spaces as well as other parameters that are asked by the program and that can affect the inside temperature and humidity. The interior of the tower was considered as an empty space with no occupancy and no artificial heating and ventilation. The effect of human occupancy has been neglected due to the fact that the space doesn’t have any real function and that it is most of the time closed to the public.

### 6.1.2 Analysis of the results

The simulation run produced results for one typical year with a resolution of half an hour. The values obtained for the level 6 and 7 (see Table 9) corresponding to the concrete and to the masonry in the clock part were analyzed.

In the two graphs below the time history of the temperatures in the external and internal surfaces of the four walls is shown.

As we can see in Fig. 92, all the interior temperature of all the walls was nearly the same. On the contrary, Fig. 91 shows that the west wall experienced the largest variation of external temperatures. Thus it was selected for the further detailed analysis.

<table>
<thead>
<tr>
<th>Thickness [m]</th>
<th>Conductivity (k) [W/(mK)]</th>
<th>Specific Heat [J/(kgK)]</th>
<th>Density [kg/m³]</th>
<th>Emissivity</th>
<th>Solar absorptance</th>
<th>Visible absorptance</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete floor1</td>
<td>1.2</td>
<td>2.3</td>
<td>1000</td>
<td>2300</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>SUPERFICIAL LAYER (cement screed)</td>
<td>0.3</td>
<td>1.4</td>
<td>650</td>
<td>2100</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>concrete floor2</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>CEMENT/LIME PLASTER</td>
<td>0.35</td>
<td>2.3</td>
<td>1000</td>
<td>2300</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>SUPERFICIAL LAYER</td>
<td>0.15</td>
<td>1.4</td>
<td>650</td>
<td>2100</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>concrete floor3</td>
<td>0.02</td>
<td>0.8</td>
<td>840</td>
<td>1600</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>CEMENT/LIME PLASTER</td>
<td>0.25</td>
<td>2.3</td>
<td>1000</td>
<td>2300</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>SUPERFICIAL LAYER</td>
<td>0.1</td>
<td>1.4</td>
<td>650</td>
<td>2100</td>
<td>0.9</td>
<td>0.6</td>
</tr>
<tr>
<td>Timber roof1</td>
<td>0.1</td>
<td>0.1</td>
<td>1000</td>
<td>500</td>
<td>0.9</td>
<td>0.78</td>
</tr>
<tr>
<td>TIMBER BOARDS (wood roofing slabs)</td>
<td>0.025</td>
<td>300</td>
<td>380</td>
<td>8900</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 11 Materials properties of the horizontal elements
As the analysis focuses on the study of the west wall the graphs related to the comparison between external and internal surface temperature of masonry and concrete belonging to this side will be shown below whereas the same considerations about the other three walls will be reported in the Annex 15 - DesignBuilder results of the walls in the 5th and 6th levels.

The concrete shows smaller difference between interior and exterior temperature as it has lower thickness and higher conductivity than the masonry as can be appreciated comparing Fig 97 and Fig 99.
The difference between the outside surface temperature of concrete and masonry is very small whereas this is more relevant on the inside surface. Furthermore the variation of the external temperature of the materials is significant compared to the one in the inside as expected.
The extremes values of the external surface temperature easily reach over 40°C during the summer and around -12°C during the winter for both masonry and concrete whereas, as expected, the inside surface temperatures are contained in a smaller range from 30°C to around -3.5°C.

| Max. ext. surface temperature concrete (°C) | 43.01 |
| Max. ext. surface temperature masonry (°C) | 44.95 |
| Min. ext. surface temperature concrete (°C) | -12.07 |
| Min. ext. surface temperature masonry (°C) | -12.95 |
| Max. ins. surface temperature concrete (°C) | 29.64 |
| Max. ins. surface temperature masonry (°C) | 29.7 |
| Min. ins. surface temperature concrete (°C) | -4.63 |
| Min. ins. surface temperature masonry (°C) | -3.28 |

Table 12 Extremes values of the inside and outside temperatures reached by concrete and masonry in the masonry and concrete

Other data obtained from the program are the air temperature and the relative humidity inside the tower, shown in the graphs below. Some observation can be done on the data obtained for the internal air: the negative value can be accepted even if in the reality temperatures may not reach the negative values. In fact the tower was modelled as a space without usage and the two big circular opening were characterized by
very low quality considering that some of the glasses composing the windows are broken having so negligible thermal properties.

![Temperature Graph](image1)

*Fig 103 Air temperature inside*

![Humidity Graph](image2)

*Fig 104 Relative humidity inside*

These data acquired have been then carefully elaborated and investigated in order to find representative intervals representative of the long and short term behaviour of the wall as will be explained in the next chapters.
7. **THERMO-MECHANICAL ANALYSIS OF THE WEST WALL OF THE TOWER**

The behaviour of a massive structural element is related to the typology of thermal load that it experiences. In general, long and short term phenomena can affect the structure and be responsible of different kind of crack patterns.

To clarify the difference between the long term and short term thermal phenomena, let us examine some typical temperature profiles of a massive wall.

![Graph showing temperature profiles](image1)

**Fig 105** Example of profiles of temperature throughout the thickness of the masonry wall studied documenting the long term phenomena

Fig 105 compares the typical profiles for summer and winter time. It is seen that, although the temperature is not uniform, there is a major change of the average temperature of the wall between the seasons. The spatial variation of temperature through the wall thickness for each of these instants is much smaller than the variation in time. This kind of long-period thermal change, which affects the entire thickness of the wall can cause stressing and damage, if the wall is constrained or if it is inhomogeneous, either through its thickness or in its plane. The latter case was studied in the preliminary analyses in chapter 5, where it was possible to see that the resulting cracks then affect the entire thickness and are apparent on both inside and outside surfaces.

![Graph showing temperature profiles](image2)

**Fig 106** Example of profiles of temperature throughout the thickness of the masonry wall studied documenting the short term phenomena
Fig 106, on the contrary, documents what we call the short time phenomena. It depicts one thermal profile with a small spatial gradient, which is achieved by slow increase of ambient temperature. The second profile has a steep gradient toward the surface, which is a result of a sudden cooling of the surface. This can cause tensile stressing only to the depth experiencing the high gradient and may result in surface cracking.

In this case the phenomena that wants to be detected cannot be considered uniform throughout the thickness of the wall and it will need a 3D model to be studied. Moreover the most representative case in which this gradient of temperature is maximum should be identify. Considering the temperature obtained from the DesignBuilder program this phenomena could be investigated using a range of temperature around the 17th of February when the minimum temperature is reached on the outside surface.

![Fig 107 Difference of inside and outside temperature on the concrete of the west wall during 10 days including the 17/2 when the minimum temperature is reached on the external surface](image1)

![Fig 108 Difference of inside and outside temperature on the masonry of the west wall during 10 days including the 17/2 when the minimum temperature is reached on the external surface](image2)

Evaluating the damage pattern detected during the visual inspection it was possible to observe that the majority of the cracks identified was visible both from the inside and outside of the wall. It is possible to suppose that this typology of crack pattern which affects the wall throughout its thickness is due to the effect of long term thermal phenomena.

In fact, the temperatures inside the thickness of the wall are changing constantly due to the continuous change of the superficial temperatures. The long term phenomenon takes into account the extremes changes of temperature to which the entire structural element is exposed from the reference temperature in
which the structure was built. This phenomena affect all the element and it is responsible to a possible crack pattern throughout the entire thickness of the wall.

In order to evaluate the temperature inside the thickness of the west wall two 1D model in Adina software representing the section of the masonry wall and of the concrete ring were considered. The real geometry of the wall layers was considered as shown in the images below and the properties of the materials considered were the same selected in the BPS program.

On the top and on the bottom the condition of no heat flux was applied. The external surface temperature history obtained with the DB program was applied on the exterior side whereas the interior side was loaded with the internal surface temperature.

For the initial conditions of the models a space function was specified for each material considering a linear distribution of temperature through the thickness defined by the initial surface temperatures; a time of 6 hours was given in order to stabilize the model.

The range of the temperatures reached by the two different wall section studied during the year is shown in the graph below.

The temperature in the interior surface and in within the wall cover a range from 0°C to 30°C in the masonry and up to 40°C in the concrete whereas the span of temperature of the area connected to the external surface is higher.
Analysis of the faults in the Church of the Sacred Heart in Prague

Fig 111 Range of temperature affecting the masonry throughout its thickness during the year

Fig 112 Range of temperature affecting the concrete throughout its thickness during the year

Since the thermal calculation carried out that the short thermal variation in the inside are very small compared to the one in the outer surface it is possible to conclude that in order to study the structure it is possible to use a 2D model and to consider the average value of the temperature throughout the thickness of the wall neglecting the extremes.

The results obtained from ADINA software were then elaborated in order to obtain the average value of the temperatures for each instant (half an hour). In this way it was possible to obtain the average time history of the temperature inside the masonry wall (Fig 113) and the concrete beam (Fig 114).
It was then necessary to establish a starting temperature corresponding to the one in which the tower was built. From the data obtained during the historical analysis and also observing the old pictures of the construction works [2], it was possible to date this period as previously reported in the chapter 2.2. In particular the construction of the tower can be dated between September and October of the 1930. In this range of time the tower was apparently built up to the clock level. Considering the results of the historical survey and the range of temperature affecting the structure during the year (Fig 111, Fig 112) two different hypothesis of initial temperature were considered: 10°C and 20°C corresponding to the 14th of October and the 1st of September.

Starting from these references temperature the minimum and the maximum temperature reached were considered and the time in which the extremes appears was checked.
For the iron profiles the average between the inside and outside surface temperature of the window was assumed as the most representative. Considering the extremely thin section of these elements it is possible to assume that the distribution of temperature within the section of the profiles is constant. Moreover this hypothesis can be supported by the fact that the difference between the outside and inside surface temperatures is very small.
temperature of the window, as calculated by the BPS program, is negligible. Then, the same considerations done for the masonry and concrete were contemplated for the temperature time history of the iron elements.

Fig 119 Average between the inside and outside surface temperature of the windows – west iron; evaluation of the extremes temperatures starting from a reference temperature of 10°C

Fig 120 Average between the inside and outside surface temperature of the windows – west iron; evaluation of the extremes temperatures starting from a reference temperature of 20°C

The extremes temperature reached by the different elements are shown in the table below.

<table>
<thead>
<tr>
<th>ELEMENT</th>
<th>REFERENCE TEMPERATURE 20°C</th>
<th>REFERENCE TEMPERATURE 10°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/9 10.30</td>
<td>20.0</td>
<td>14/10 6.30</td>
</tr>
<tr>
<td>8/8 17.00</td>
<td>29.4</td>
<td>8/8 17.00</td>
</tr>
<tr>
<td>MASONRY</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/9 10.30</td>
<td>20.0</td>
<td>14/10 6.30</td>
</tr>
<tr>
<td>8/8 17.00</td>
<td>26.9</td>
<td>8/8 17.00</td>
</tr>
<tr>
<td>IRON</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/9 21.30</td>
<td>20.0</td>
<td>14/10 11.30</td>
</tr>
<tr>
<td>8/8 17.00</td>
<td>43.0</td>
<td>8/8 17.00</td>
</tr>
</tbody>
</table>

Table 13 Load functions referred to the two different reference temperatures

The final temperature loads applied to the model are shown in the table below, where the difference in the time distribution of some hour was neglected in the model as long term analysis are performed. The first
main step (TIME 3) can be identified in the moment in which masonry and concrete reach the lowest
temperature. Then while their temperature start increasing again the iron elements will reach the lowest
temperature (TIME 4). The final step represent the situation in which the tree elements achieved reach the
highest temperature (TIME 5).

<table>
<thead>
<tr>
<th>Concrete temperature (°C)</th>
<th>Masonry temperature (°C)</th>
<th>Iron temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TIME 1</td>
<td>REFERENCE T</td>
<td>REFERENCE T</td>
</tr>
<tr>
<td>TIME 2</td>
<td>REFERENCE T</td>
<td>REFERENCE T</td>
</tr>
<tr>
<td>TIME 3</td>
<td>-6.27</td>
<td>-3.99</td>
</tr>
<tr>
<td>TIME 4</td>
<td>-4.59</td>
<td>-2.83</td>
</tr>
<tr>
<td>TIME 5</td>
<td>29.43</td>
<td>26.92</td>
</tr>
</tbody>
</table>

Table 14 Temperatures trends of the tree elements

The geometry considered in the model represents the part of the wall in between the belfry and the
connection between the roof of the Church. Starting from the model used in chapter 5 the model was
redefined. In the lower part the vertical displacements were constrained considering the wall as infinitively
stiff as its thickness increases by 60 cm and it is connected with the other part of the building. This
hypothesis was also verified in the initial model checking the magnitude of the vertical displacements at the
connection level. Free horizontal displacement was allowed along the bottom line except for one point at at
its center. Moreover 8 rigid links were considered in the connection between the concrete beam and the iron
representing the possible links partly detected during the visual inspection partly assumed symmetric to the
ones detected.

The missing part of masonry corresponding to the belfry and to the floor connected was reproduced applying
a pressure equal to 0.02 MPa on the top of the masonry.

The non linear characteristics considered for concrete and for masonry are shown in the Table 8 whereas
for the iron profiles a linear elastic behaviour was considered assuming that the stresses magnitude will
always be under the yielding point. The non linearity materials were implemented in the ADINA software
using the Concrete Material Model as explained in chapter 6.

The masonry wall was modelled considering 2D solid plane stress elements with the corresponding
thickness of the wall equal to 1.2m. In the same way the concrete was represented by the same elements
with a thickness of 60 cm. The iron beams were modelled using Hermitian beams elements and specifying the properly cross sections (see chapter 5).

Result of the calculations are presented in the form of the contour band plots of the maximum principal strain (which can be associated with crack width) and plots showing the position and direction of cracks.

Fig 122 shows that cooling of the structure from the initial temperature of 10°C to the lowest temperature of concrete and masonry (TIME 3) results in formation of small radial cracks in the concrete ring (mainly at the 12 and 6 o'clock position) and some tangential cracking along the concrete-masonry interface (between 5 and 7 o'clock position). When the iron frame reaches the minimum temperature at TIME 4 (Fig 123), additional diagonal cracks form in the concrete near the connection of the horizontal iron beam (3 and 9 o'clock position). Subsequent increase of temperature to the highest state (TIME 5, Fig 124) results in extensive cracking of the masonry. The contour plot shows that most cracking localizes along the vertical lines at the 12 and 6 o'clock positions, but some radial cracks between 11 and 1 o'clock positions also develop. Due to the fact that the vertical iron element is pushing up the concrete the tensile strain in concrete at 12 o'clock increase. It is also possible to notice that the tangential cracks along the interface and the tensile cracks near the attachment of the horizontal cross beam closed.

Fig 122 Principal strain and cracks plot for the time 3 considering a reference temperature of 10°C

Fig 123 Principal strain and cracks plot for the time 4 considering a reference temperature of 10°C
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ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Fig 124 Principal strain and cracks plot for the time 5 considering a reference temperature of 10°C

Compared with the results obtained starting from a reference temperature of 10°C having as initial temperature 20°C involves a bigger cooling down phase. Comparing Fig 125 with Fig 122 (both of them referring to the TIME 3, i.e. they show the effect of the cooling of the structure from the initial temperature to the lowest temperature of concrete and masonry) it is possible to observe how the bigger cooling of the structure is proportional to a bigger crack pattern. In addition to the considerations done for the Fig 122 two radial cracks can be appear in the concrete ring corresponding to the 4 and 8 o'clock positions. Furthermore a severe damage pattern has already appeared in the connection between the concrete ring and the horizontal iron profile whereas in the previous case its presence was appreciable just from the TIME 4. Finally a more spread crack pattern is present in the masonry over the concrete ring at the top of the opening.

Fig 125 Principal strain and cracks plot for the time 3 considering a reference temperature of 20°C

When the iron frame reaches the minimum temperature at TIME 4 (Fig 126Fig 123), additional circumferential cracking appears along the concrete-masonry interface around the 3 and 9 o’clock positions.
The warm up or the structure up to the maximum temperature corresponding to the TIME 5 shown in Fig 127 causes expansion of the concrete ring which results in the closure of the existing cracks. Furthermore, the masonry expands less than the concrete which results in the opening of new cracks at the 11 to 1 o’clock position. When we compare this cracking to the one obtained when the initial temperature was 10°C we see that the cracking is much less extensive. This is due to the fact that the difference between the initial temperature and the temperature at TIME 5 is less in the case in which 20°C was assumed as initial temperature.

In conclusion, comparing the results obtained from the numerical models shown in Fig 122 - Fig 127 with the damage detected during the visual inspection shown in Fig 128- Fig 129 it is possible to observe that the results confirm the adopted assumptions and the hypothesis that it exists a correlation between the detected damage pattern and the thermal loads affecting the wall.
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In particular, due to the different thermal properties of the materials, the cooling down of the structure generates cracks in the concrete in the radial direction. These are almost symmetric and, even if it was not possible to have a detailed survey of the exterior cracks, from the damage identification detected it is possible to see how cracks in the concrete appears inside and outside in the same locations. The concrete presents the highest concentration of stresses in the areas connected with the iron cross due to the higher thermal expansivity and higher elastic stiffness of this material. In the connection between the horizontal iron profiles and the concrete ring cracks develop forming an arch shape detected in this area during the visual inspection as shown in Fig 130.

The expansion of the structure due to the increase of temperature is responsible of the cracking in the masonry. This is mostly concentrated on the top of the opening following the radial direction at 11 to 1 o’clock positions as detected during the visual inspection (see Fig 128). At the bottom of the opening another crack line starting from the concrete line progress in the masonry following the radial direction (6 o’clock).
Tangential cracking develops along the concrete-masonry during the time due to the different expansivity and stiffness of the tree materials and to the not synchronized increase and decrease of temperature between concrete and iron.

In general the cooling down is the most probable cause of cracking in the concrete in radial direction and in the boundary between concrete and masonry; whereas the warming up of the structure can cause cracking in the masonry mostly concentrated at the top or at the bottom of the opening in radial direction.

In conclusion, it is possible to affirm that the results obtained with the two models considering two initial different temperature were together able to represents the crack pattern detected during the visual inspection. The analysis with initial temperature equal to 10°C was more precise in the characterization of the crack pattern in the masonry. In fact the final principal strain plot shown in Fig 124 identifies the three lines of cracks developing in the masonry in radial direction at 11, 12 and 1 o’clock positions and the main one at the bottom of the opening at 6 o’clock position. The second model with initial temperature of 20°C was more precise in the characterization of the crack pattern in the concrete ring and in the boundary between concrete and masonry.

This fact can be attributed to the fact that the structure can have experienced during its life bigger fluctuation of temperature than the one obtained from the Design Builder simulation for a typical year and so the higher and lower temperature can have been higer than the one considered.

It is so important to highlight that the long term phenomena detected can be further evaluated with analysis considering several years. In fact, over many years the cracks experience cycling opening and closure which leads to accumulation of the damage.
Analysis of the faults in the Church of the Sacred Heart in Prague
8. RECOMMENDATIONS FOR FUTURE INTERVENTIONS

Although the design of intervention is not included in the purposes of this work some recommendations can be done in the light of what was discovered.

The result of the damage identification establishes the absence of damage which could affect the stability and safety of the structure. The only two severe crack pattern that can be relevant regarding the structural behaviour of part of the building were detected in the tower.

However, several non structural damage were identified in all the building. The agents causing the damage should be identified and specific interventions for each one should be applied in all the areas. A detailed maintenance plan should be prepared. This should ensure the preservation of the building and the enhancement of its original composition. The different areas of the building should be treated in the most appropriate way considering their material, function and usage.

For the structural damage identified in the part of the walls around the circular openings of the tower several steps should be performed in order to define the most suitable intervention. A possible intervention should take into account the thermal movement that the different materials will experience during the future. This behaviour in situations in which the element is free is not dangerous but it can become the cause of damage when the movements of one material are constrained by the nearby one.

In this perspective, the iron elements can be detached from the concrete ring and a low stiff layer able to absorb the related displacements can be positioned in between. The iron profiles should be connected to the concrete ring in such a way to transmit wit safety the out of plane forces due to the wind. So the bolts have to prevent tangential displacement but to allow the radial one. Also the connection of the horizontal iron profile with the concrete ring should be analyzed and the sliding in axial direction should be not constrained.

Another kind of consideration should be done for the design of the intervention on the cracks appearing on the concrete and on the masonry. A possible intervention can be the implementation of a new layer of plaster. This should be able to receive the movement of the cracks behind as the wall will be always exposed to thermal changes. This can be done allowing some space between the plaster and the cracks using fabric or fibres. These contribute in strengthening the plaster and to accommodate the movement of the cracks in a larger area. It is recommended to do this intervention during the period in which cracks are closed, in fact although the expansion is supported by this mechanism the opposite movement will produce the detachment of the mortar at the crack location. Furthermore, it is strongly recommended to avoid interventions related to the filling of the cracks. In fact, even with the use of extremely soft material the continuous opening and closing of the cracks will make this intervention not durable.
Analysis of the faults in the Church of the Sacred Heart in Prague
9. CONCLUSIONS

The aim of this work was evaluating the conditions of the Church of the Sacred Heart in Prague. Especially in the cultural heritage, where data, materials and technologies can be unknown the importance of the use of each tool available and recommended in the literature, in the proper order, is the key for a successful evaluation of the building. Considering the constant multiplicity that characterize the historical patrimony and its analysis the importance of a rigorous procedure for the evaluation of the building will ensure the completeness of the work.

In particular, in this study the historical analysis carried out important data for dating the construction of the tower and so for the assumption of a reference temperature. On the other hand it was fundamental for the understanding of the cultural value of the building which should be preserved in every future rehabilitation works that will be performed on the building. This is related to the artistic, spiritual value as well as to the cultural identity that it is representing.

Through the visual inspection tool it was possible to collect information for defining the entity of the damage affecting the building. In particular, the analysis carried out non structural damages related to moisture and to the thermal movements caused by differing rates of expansion or contraction between the components. The most significant damage detected appeared on the main longitudinal walls of the tower where a crack pattern develops around the two circular openings.

The aim to investigate the thermal behaviour of the massive walls of the tower was studied with an original approach due to the lack of a monitoring plan for detecting the thermal and physical conditions. In fact a BSP program was used instead of monitoring. This advantageous choice produced reliable results that were then evaluated and used for the structural analysis of the tower. After the appropriate consideration and simplification of the enormous amount of data obtained, the analysis was able to identify the long term thermal phenomena as the main cause of the crack pattern identified during the visual inspection.

A first step for further analysis also considering possible future intervention should be a monitoring of the thermal condition and also of the actual state of damage. The monitoring of the cracks in the walls can turn out if the movement of the cracks is still active. This together with a more detailed survey and representation of the existing cracks should be enough to represent the actual condition and to be able so to understand the upgrade of the damage.

Furthermore, the monitoring of the temperature on the interior and exterior of the tower can produce a real input data for verifying the BPS output and eventually for a more detailed numerical analysis. In fact, the analysis performed in this work have been done for a typical year, while the most severe damage will be caused by extremes that the historical building experiences. With this data additional analysis can be performed investigating for example the behaviour of the wall under cycling long term phenomena.

Moreover, other analysis should be considered if the damage on the external surface of the masonry wants to be evaluated. In fact, short term phenomena can be evaluated on a 3D model of the wall in order to discover the extent of the damage that these can produce.
Another study that should not be missed in future analysis of the building is the evaluation of the condition of the ramp. For the characterization of the unknown morphology of this element a testing phase is strongly recommended. For this purpose GPR tests can be used to discover the composition of the ramp and the eventual presence of steel elements inside. Thereafter, the characterization of the materials can be deducted from the other tests already performed or can eventually be evaluated with some minor destructive test or with the extraction of some core if needed. However the use of DT should be limited to a specific point in which the morphology was previously detected with other tests.
10. REFERENCES


