Title:

STRUCTURAL EVALUATION AND INTERVENTION PROPOSALS FOR SANTA MARIA DEL PI CHURCH

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Date:

March, 2009
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1. INTRODUCTION

The Church of Santa Maria del Pi is an important example of the Catalan Gothic architecture in Barcelona, Spain. The building dates from the Middle Age and has a long construction history with additions, repairs, etc.

Within the scope of this report, to identify the causes of damage and decay, the history of the building has been studied in the integrity of all its components as a unique product of the specific building technology of its time and place. The understanding of the history of this particular building has been a parameter to recognize the original construction techniques, the changes it has suffered through the time and the historical actions carried out in past stages.

As heritage structures requires organization of studies and analysis; condition surveys, identification of the cause of the damages and decay, as well as the choice of remedial measures and control of the efficiency of the interventions have been carried out.

A detailed damage survey was performed during the inspection stage in order to understand the condition of the materials and the structure at present state as well as decide the immediate measures to be undertaken. The building was divided in three different parts each one of them corresponding to the façades (“Main façade, “Evangeli” façade and “Apse” façade). Damages were defined as “non – structural” or “structural” and mapped on Autocad drawings with certain legends.

Safety evaluations have been done to determine the acceptability of the safety levels by analyzing the present conditions of the building and its materials. Characterization and validation of typical failure modes have been realized to assess these conditions.

Monitoring and non – destructive tests as part of the intervention proposed have been considered to take decisions on any further actions. The proper techniques will be described in the correspondent chapter.

The intervention proposed has been taken into account within the context of the restoration and conservation of the whole building with special attention on respecting the characteristics of the structure, as well as the original concept, material and construction techniques based on truly durable and compatible materials as well as in its reversibility, removability and non – intrusiveness.
2. HISTORY

2.1. Localization

The church of “Santa Maria del Pi” is located in the ancient part of Barcelona known today as the “Pi neighborhood”, where mostly merchants and the oligarchy lived in the ancient times; the neighborhood was settled during c. XIII by the construction of the new city wall. (Figure 1)

This edification is composed by structures of different chronologies between which we can find; the church itself, bell tower, the chapel of “Sang” and the rectorate which are dated from the early medieval period.

The building is limited, on the north and west by Carrer Cardenal Casañas, on the northernmost part by Plaza del Pino, on the east part by Plaza Sant Josep Oriol and on the southernmost part by Plaza del Pino. (Figure 3)

Figure 1 – Partial plan of the city of Barcelona
(Ciutat Vella)

Figure 2 – Partial plan of the old neighborhood
with the localization of Santa Maria del Pi church

Figure 3 – Partial plan showing the limits of Santa Maria del Pi church
3. DESCRIPTION OF THE BUILDING

El Pino, as a religious building from the latest gothic (the construction of the church is known to be finished by the end of the 15th century), has a basilical plane with a single nave of 55m length, 16m width and 27m height. It has lateral chapels between its buttresses and a polygonal apse on the southwest where two more chapels are located at each side of the presbytery, a sacristy and a main stair to enter the bell tower and the roof. The nave presents an internal organization of seven rectangular sections of approximately 6m each one covered with quadripartite vaults; whereas the apse and the presbytery are covered with a palm-shaped ogive vault.

The toral and ogival arches of the roof of each section of the nave rest on the columns inside the buttresses separating the chapels. From the level of the impost these capitals form the cornice that crosses the whole perimeter of the church and from which the windows of the cloister go out.

It is important to mention that the internal image of the church has been the consequence of diverse restorations since the end of its construction (end of XV century).

The building consists of four facades that will be described in a particular way in this report and they will be identified as follows (Figure 9.b.)

![Figure 4 – General plan of Santa Maria del Pi church](image)

Figure 4 – General plan of Santa Maria del Pi church
3.1. “Main” Façade

The “Main” façade, which has not been completed until 1497, is composed by two parts. The superior, with a great rose window of an overall length of 10m in its center and the inferior defined by a serie of arches forming the portal and flanked by two polygonal towers. It is 27.40m length and 30m height.

The major portal who stands in front of lead is defined by the presence of arquivolts in gradual change with vegetal decoration crowned by a rosette. The rest is divided in two parts by a cornice. The entire set shows blind arcades on both sides, with few sculptural ornamentation.

At the center, the tymanum presents three arches forming little chapels which give shelter to the sculpture of the “Virgin”.

In the superior part of the façade a rose window is observed - very much in line with the Gothic style of the church - with an overall diameter of 10m. It is a faithful reconstruction of the original one, destroyed during the civil war (also reconstructed in 1721).
3.2. “Evangeli” Façade

From the “Evangeli” façade facing the San Josep Orial square, it is necessary to stand out the “Ave Maria” portal considered the most ancient of all the building defined by arquivolts in gradual change occupying part of the interior space of the buttresses of the church. Each one of this arquivolts rests on columns which presents decoration of zoomorphic and vegetal elements.

The Evangeli façade of the church is 47m length and almost 30m high. It’s including 8 buttresses with a medium thickness of 1.5m. On the central part there is the lateral door of the church which is 3m width and 3.75m high.

The façade shows a level in which we can find a basement along the base of the buttresses which is used as a frame for the walls that close the chapels. In the central part of these walls, windows crowned by ogival arches are present.

3.3. “Epistola” Façade

This façade is 48.40m length and almost 30m height. No further description of this façade was possible to carry out because of the buildings surrounding it.
3.4. “Apse” façade

The physiognomy which nowadays we can see of “Apse” façade is consequence of the restoration made at the end of the XIX century. It is composed by a portal of simple lines defined by a pointed arch and a tympanum sticking out from the façade and supported on impost. As this façade lacks from the presence of chapels, the buttresses can be seen from its base. As a consequence of this characteristic, the perimetral circulation through the apse has to be done by some reduced vaults.

3.5. The roof

The main characteristic of the roof is its physiognomy of slopes without sharp angles. From the culminating points, the sections of the roof evolve longitudinally in a waved plane that is adapted laterally to the slopes. The complex is realized by means of a crossbeam adapted to the form of the roof and formed with tile barks and bricks. As the chapels, it has openings used for ventilation covered with stones of different measures.
Another important element is the structural configuration in the apse, which principal aim is to support its key. It is made of an elevated construction defined with a central core of conical form, from which eight radial elements start and that coincide with the nerves of the lower ogival vault. The nerves are small platforms covered with Portland mortar with a network of iron beams supported in two legs.

![Figure 11 – Roof configuration](image)

### 3.6. Intervention Works

The church is known to be built on the site of an earlier Romanesque one which was supposedly used for propping the actual church. The first information regarding construction is dated from 1322; however the official news are only from the second half of the 14\textsuperscript{th} c. According to these records, the church was finished by 1391 and the façade overlooking the Plaza del Pino could only be completed in 1497; with the bell-tower being added between 1460-1486.

The main interventions to the building are dated to first quarter of the 16\textsuperscript{th} c. when the altar was renewed. The crypt was built in the 1570's, but was abandoned during the 18\textsuperscript{th} c. due to floods and deterioration. During the Succession War (1714), the church suffered several damages and the most important of these was the explosion of a powder store near the building which caused serious damages in the vaults, the windows, the walls, the pavement and the rose window. Between 1718-1721, a restoration program was launched, concerned mainly on the repair of vaults, the lateral walls and the rose window. Another restoration was carried out in the 19\textsuperscript{th} c., to remove all Baroque decoration together with the pavement of the chapels and the nave. Also during these works, an auxiliary structure to support the keystone of the central vault was erected and some timber members were replaced by steel ones.

Still in the 20\textsuperscript{th} c., some important restorations were carried out, mostly after the Civil War, in the 1950's: The rose window was rebuilt following the original model; most deteriorated stones were substituted; open cracks were repointed and a new chorus was built (Fig.4 and 5). Thus, the actual appearance of the church is a result of successive interventions during its long history.
3.7. Building Process

Apparently there was no initial architectural project that could help to define some of the most relevant processes during the construction of the church, in fact, when some comparisons are made to the different plane distribution of the apse, the variability in the measures and the element distribution is excessive, possibly related to the different time periods of the building process or the restrictions of space that the site had because of the presence of a previous Romanesque church.

The walls that close the lateral chapels prove the lacking in physical unity with the buttresses that flank them indicating that they might be constructed in an independent way from a chronological point of view. However, a closer look to the façade let distinguish the presence of a continuous basement which contradicts the previous statement as the base of the buttresses and the basement are the same. By the other hand, the relationship between the foundations and the basements gives prove of a single architectonic structure. The previous evidences let deduce the construction process of the first levels of the church which indicates, that the foundations and the basement were continuously built. The apse side is more complex to understand while the measures of the buttresses change. All the above mentioned features implied that as a first step, the perimeter walls and buttresses were built together until reaching a certain level. Then, the remained walls from the apse towards the main façade were constructed while at the same time, the walls from the chapels were built and the church was incrementally closed.
4. MATERIALS

The stone with which this church was constructed comes from the “Montjuic” quarries, corresponding to a silicic stoneware which characteristics gives it resistance to chemical attacks and high durability. In general, the stones forming this religious building are of good quality in composition and granulometry.

The mortar is formed by lime of fine granulometry flattened in order to limit the entrance of water. In some modern restitutions, it seems that mortar of fast cement, has been used for the pink color presented on the joints.
5. STRUCTURE

5.1. Soil Conditions

The soil beneath the structure is composed by different layers. During the construction process, the different states of the soil caused differential settlements between the lateral walls, as well as the buckling of the façade and the buttresses. However, these settlements seem to have been stabilized.

The first layer composed by man-made filling of poor condition with a thickness between 2.00 and 2.60m, the second one made of clay and sand with a thickness between 4.00 and 4.50m and the last one of low permeability, high expansion potential and high deformation with a thickness between 5.20 and 12.80m.
5.2. Foundation

Some inspections have been made to determine the conditions of the foundations. However, as it has not been possible to go deeper on the excavations made, some hypotheses have been developed. The first one assuming that the foundation is at 2.00m depth over the human made filling and the second one supposing that the foundation is at 3.00m depth over the clay layer. It seems to be formed by series of continuous walls with the same width attached by lime mortar. The excavations performed show that the main façade foundation is continuous with the one of the lateral walls formed by shows rows of stone blocks well connected until the surface level.

![Figure 20 – Picture illustrating the different layers of the soil](image)

5.3. Structural Elements

5.3.1. Buttresses and walls

They are composed by a three leaf arrangement with two external layers of ashlar masonry and an internal core made of rubble masonry having a good connection. The external leaves are rather thin due to the dimension of the stone blocks used reducing the stiffness.

![Figure 21 – Picture illustrating the different layers of the soil](image)
5.3.2. Arches and vaults

These elements are constituted by dowels of ashlar masonry of more dimensions than the ones used in the walls.

One important characteristic that should be mentioned is the composition of the nave vault which is formed by a medieval rubble material of variable height. In its top a continuous crossbeam adapted to the shape of the vaults is supported by an air chamber between series of brick walls which has a layer of rubble composition underneath.

![Figure 22 – Picture illustrating the characteristics of the vaults](image)

6. DAMAGE SURVEY

Within the scope of this report, the damages of the main, lateral and apse façades were studied. Damages were defined as “non-structural” and “structural” and mapped on AutoCAD drawings with certain legends. Possible causes for damages were listed.

6.1. Main Façade Visual Inspection

6.1.1. Methodology of the inspection

In examining Santa Maria del Pi church, the façade was inspected for signs of damage, both structural (Figure 23) and non-structural (Figure 24). The observed cracks, both open and repointed were considered as structural damage; whereas vegetation and evidence of moisture were considered non-structural. In addition, for the purpose of organizing an inspection checklist, the main façade was divided into east tower, west tower, the area above the rose window, the area at the same elevation as the rose window, and the area below the rose window.

![Figure 23 – Picture illustrating the main façade](image)

1. Material made of tile
2. Air chamber
3. Rubble material

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1 No possible visual inspection was carried out on the “Epistola” façade because of the buildings surrounding it.
6.1.2. Geometry and materials

The Main façade of the church is composed by two principal bodies. The inferior is defined by a serie of arches forming the portal and the superior is caracterized by the presence of the rose window with an overall diameter of 10m. It is 27.37m and 30.50m. The main facade as almost all the Santa Maria del Pi church, is mostly built with local Montjuic stone and lime mortar.

6.1.3. Structural Damage

On the Main façade, most of the cracks seen were step-like cracks along the mortar joints. Some had been repointed and remained so, but others had been repointed and reopened again. In the areas near the ground that were easily accessible, some cracks were not fully open but the mortar crumbled easily to the touch (Figure 23).

6.1.4. Non – structural damage

Evidence of non-structural damage—vegetation, moisture and some compatibility cracks—could also be found on the main façade. Vegetation was seen in the form of moss and plantlife. The vegetation was rooted both in the mortar joints of the masonry (at high and low elevations) and also between the shingles on the roof of the entrance. Furthermore, there was evidence of moisture in the stone, visible by discolorations in the stone and decorative elements of the façade. The stone had a dark patina in some areas. Also, on the entranceway, the upper elements were discolored white, possible evidence of efflorescence (Figure 24).
6.1.5. Hypotheses for damages

The cracks appearing on the façade can be identified as structural and non structural depending on their behaviour. The historical surveys as well as the soil conditions have been useful in order to understand the changes the church has suffered through its construction and relate them to the damage that has been possible to identify.

It is known that the rose window as well as the upper part of the façade suffered some damages due to an explosion and were completely reconstructed on 1940. The diagonal cracks appearing on the top of the rose window (Annex 1 Picture 5.a.) can be attributed in that case to soil settlements as this part due to the restitution of material has become a weak area with poor connection.

On the other hand, the diagonal cracks identified on the lower part of the rose window (Annex 1 Pictures 8.a. and 9.a.), can be classified as compatibility cracks; most of the buildings with these rose windows present the same kind of cracks, which can be easily explained as a consequence of lack of loading on that area which tends to separate from the lateral part submitted to higher compression forces.

The cracks inspected near the east tower (Annex 1 Pictures 5.a., 7.a., 15.a. and 17.a.) can be related to the different movements the church has experienced due to soil settlements. From the visual inspection it was possible to see that these cracks have been refilled with a different kind of mortar presenting a different color. Not all of them have re–opened; cracks F – 1c and F – 3c are presenting this behaviour while crack F – 10c is apparently stabilized, reason why it is possible to think that the settlement has stopped. However, special attention must be paid.

The appearance of cracks on the gothic decoration next to the door have been related to the leaning of the façade during past years. It is possible that the difference in movement between the upper and the lower part of the façade have been the principal cause. Further monitoring on these cracks must be carried out in order to know their real depth and if their movement is developing or stabilizing.

6.1.6. Conclusions

Since soil settlements are the main cause of most of the damages presented on the façade, special attention must be paid regarding this issue. Special proposals must be suggested to monitor the behaviour of the soil as well as detailed inspection of the foundations. Also, special attention to the different movement between the towers and the façade is important to take into account and necessary to monitor constantly. Some recommendations must be proposed in order to assure the safety of the building and take the adequate actions in case it is needed.

Less dangerous but not less important must be the recommendations for the non structural damage. Some actions must be taken to avoid the growth of plants and appearance of moss. Suggestions for the removal of black patinas can be carried out.
6.2. “Evangeli” Façade Visual Inspection

6.2.1. Methodology of the inspection

For the visual inspection, carried out on this façade, an Auto-cad drawing provided earlier was used. In order to organize better the visual inspection of the façade, it was divided into three parts (Figure 25):

- **Right part**, which start from the corner were the main façade is connected with the lateral one, and includes the first three buttresses.
- **Central part**, which is mainly characterized by the lateral door and the two towers. The right one, which is what remains of a small and ancient bell tower, and the left one.
- **Left part**, which includes the last two lateral buttresses, and arrives until the connection with the apse.

![Figure 25 – Methodology for damage survey](image)

6.2.2. Geometry and materials

The lateral façade of the church is 47m length and almost 30m high. It's including 8 buttresses with a medium thickness of 1.5m. On the central part there is the lateral door of the church which is 3m width and 3.75m high. The lateral façade, as almost all the Santa Maria Del Pi church, is mostly built with local Montjuic stone and lime mortar. In the external part the stones are cutted with a rectangular shape.

6.2.3. Church’s corner

Looking at the right side of the lateral façade is possible to see the presence of two different types of stone along the vertical direction of the church’s corner. It is probable that this kind of intervention was
done in the past when some intersection problems between the connection of the lateral and main-
façade were found. However, nowadays the connection is not good. The typology of the intervention
using different stone sizes and its not homogeneous distribution doesn’t help the structure to reach a
good behaviour (Annex 2 Picture 2.b.).

6.2.4. Church’s basement

Checking the church morphology of the “Evangeli” façade, it is possible to see a new stone’s layer,
probably added later when the first soil settlements appeared, with a 1.30m height and 17.00m length
with a maximum thickness of 0.35m starting from the right corner up to the main door (Annex 2 Picture
4.b. and 5.b.).

6.2.5. Right Part

A) Non – structural damages: Loss of material

On the right side of the wall, in the bottom part of the corner (Annex 2 Picture 4.b. and 6.b.), it is
possible to find an area with a distributed loss of mortar probably caused by human activities and
weathering attacks. Several voids can also be found in this area probably removed to make pass
some light cables.

B) Structural damages: Cracks on left side

A crack of considerable dimension can be observed on the left upper section of the lateral wall
progressing until reaching the lower part. By the interpretation of the appearance of this crack it is
possible to make hypotheses on its structural performance and understand the church’s tendency to
separate in two different parts. (Annex 2 Picture 1.b.).

6.2.6. Central part

A) Non – structural damages: Environmental Attacks

In the central area of the lateral façade it is possible to find some plants growing along the buttresses,
mainly in correspondence of the gargoyles. In the same areas as expected, it can be found damages
caused by the effects as leaching and humidity. (Annex 2 Picture 20.b.)

B) Structural damages: Main door and buttresses

In this section of the façade it can be noticed how the geometry of the two buttresses has not the
same width. Checking from the top to the bottom it can be seen how the thickness of the buttresses in
the two lower meters is smaller than the one on the superiors. To explain the apparently movement
that the right side of the door's arch is experiencing it is possible to consider that part of the weight coming from the upper buttresses are acting on the arch of the door. (Annex Picture 20.b.).

6.2.7. Left part

A) Structural damages

In this part of the building is possible to notice a crack of considerable dimensions on the left side of the structure, which starts from the bottom of the window and crosses the wall until the floor. A probable movement of the apse can explain the appearance of this crack and the behaviour of the entire structure. (Annex 2 Picture 21.b.)

B) Hypotheses for damages

For the structural damages found in this area it can be stated that they are shear and diagonal cracks. The appearance of these cracks can be connected to differential settlement between the lateral wall and the buttresses that has occurred in the past.

C) Conclusions

Most of the structural damage that has been observed at this facade is damage due to settlement that has been developed in the past and does not continue to occur any more. In contrast with this, the non structural damage, such as vegetation and humidity, is predominant at the facade, due to lack of maintenance of the church.

6.3. Apse Façade

6.3.1. Methodology of the inspection

For the visual inspection, carried out on the apse of Santa Maria del Pi Church, an Auto-cad drawing produced by the team members, relying on the plan and lateral façades of the building was used\(^2\).

During the inspection, the walls (1-5) and buttresses (A-F) of the apse were divided into consecutive parts and studied separately (Annex 3, Fig. 2.c.). In each part, original structure and later additions; non-structural and structural damages were mapped on the above mentioned Auto-cad drawing (Figure 26). However, it was not possible to carry out any measurements because of the barriers placed for the restoration works. An extensive phographic survey was also carried out (Annex 3).

\(^2\) Since this part of the church was not included in the survey, some mistakes in dimensions should be expected.
6.3.2. Geometry and materials

The apse of the church has a pentagonal form, with buttresses protruding from the main walls (Annex 3, Figure 2.c.). On the lower levels, there are later additions, surrounding the structure. The apse is mostly built with local Montjuic stone as the rest of the church, using a kind of lime mortar; occasional repairs with bricks and many repointed sections can be observed. The additions are mostly built with the same stone; however, one of them is built with bricks and plastered with cementitious material.

6.3.3. Walls (Wall 1 – 5)

The construction and materials of the “walls” can be considered quite homogeneous throughout the apse (Annex 3, Pictures 3.c., 4.c., 10.c., 15.c., 20.c.). Blocks are mostly rectangular and are not very thick. Around windows and doors, bigger ashlar blocks have been used (Annex 3, Pictures. 13.c., 15.c., 18.c.). It is very difficult to distinguish the original lime mortar due to losses and repointings with different colours and textures. The walls have a thickness of approximately 58cm on the upper levels, which can be considered quite thin for a height of almost 30m³.

The annex buildings on the lower levels have different characteristics and materials. One part is built with the same stone but of larger and more regular blocks (Annex 3, Picture 5.c.). Another part is built with bricks and plastered with cementitious material (Annex 3, Picture 10.c., 11.c., 14.c.). There are also parts showing mixed masonries (Annex 3, Picture 21.c.).

---

3 These values are taken from the provided Autocad drawings. Thickness on the lower part is not shown due to the annexes on this side.
6.3.4. Buttresses (Buttresses A – F)

Buttresses mostly show a similar construction with the walls, with the same kind of stone and mortar (Annex 3, Picture 27.c. and 35.c.). Also on buttresses, occasional repairs with bricks can be observed and some parts are again pointed.

The thickness of the buttresses varies between 1 - 1.8m⁴ and they protrude between 2.09 - 5.98m⁵ from the walls. Buttresses A and F are known to be enlarged during history and this can be seen clearly with a continuous joint between the older and added parts (Annex 3, Ph. c. 27.c. and 35.c.). Their length is approximately 25.5m at the back (where they meet the walls) and 22.5m on the front.

6.3.5. Damages

A) Non – structural damages

The main non-structural damages on the façades (walls and buttresses) are vegetation (Annex 3, Picture 5.c., 15.c., 19.c. and 21.c.) and black patinas (Annex 3, Picture 8.c., 10.c. and 12.c.), both of which seem to be resulting from excessive moisture. Moisture can also be accepted as the reason of loss of mortar in many places. These two phenomena (vegetation and loss of mortar) help each other’s progress and create serious problems.

Birds nesting in the holes on the walls⁶ are yet another source for damage as their excrements have an acidic composition which can deteriorate stone blocks and also present a bad appearance.

B) Structural damages

Interestingly, a similar crack pattern can be seen on all façades. This is comprised of stepped, diagonal cracks over and under the windows (Annex 3, Figure 2.b.; Annex 3, Picture 8.c., 9.c., 12.c., 13.c., 19.c., 24.c. and 25.c.). Some of them are more distinguishable, while others seem to be repointed and are difficult to recognize.

Two long diagonal cracks can be observed in the middle portion of Wall 1 (Annex 3, Picture 6.c.). These can be followed all through the wall length and also on the annex building. Also on this wall, the arch of the small “bridge” seems to have developed some hinges (Annex 3, Picture 8.c.). It can clearly be seen that, some part of the wall below the “bridge” have been reconstructed at some point in history.

The upper portions of Wall 2 present the same kind of damages as Wall 1 (Annex 3, Picture 13.c.). But curiously, a horizontal crack can also be observed on the top part of the annex (Annex 3, Picture 11.c.).

⁴ Buttress A: 1.5 m; B: 1.0 m; C: 1.7 m; D: 1.77 m; E: 1.81 m; F: 1.61 m.
⁵ Buttress A: 4.46 m; B: 2.09 m; C: 2.39 m; D: 2.1 m; E: 3.78 m; F: 5.98 m.
⁶ These are supposed to be scaffolding holes.
Over the window on Wall 3, there are some partial reconstructions with brick, which indicates a preceding failure (Annex 3, Picture 19.c.). The wall surface on this part is somewhat not regular, which also possibly is pointing out to repairs. Repairs with brick are also present on the two neighboring buttresses (Buttress C and D). The lower part of this wall seems to be a later addition, due to its peculiar relation with the above mentioned buttresses (Annex 3, Picture 15.c.). On the right side of the door, ashlar blocks have lost most of their mortar and appear to be moving out of their places (Annex 3, Picture 17.c.). Next to this part, the left side of Buttress C also seems to be highly stressed, evident by a continuous longitudinal crack and loss of material on the lower part (Annex 3, Picture 17.c.).

Similar phenomena can be observed on Walls 4 and possibly on 5 which was impossible to examine because of the additional buildings on this part of the apse. Yet, one of the annex buildings on this side has large cracks located around its openings (Annex 3, Picture 26.c.).

C) Hypotheses for damages

As has been stated above, moisture seems to be the most important reason for the non-structural damages (vegetation and black patinas). The source for moisture is probably defective gutters; and inadequate inclinations of some parts that help retain water (Annex 3, Pictures 13.c. and 16.c.).

Another possible reason for the black patinas -that works in collaboration with water- should be air pollution, since the church is located in the very center of the city. However, it was observed that, these patinas are more concentrated on some parts of the building -such as the decorations over the windows and the cornices (Annex 3, Pictures 9.c., 19.c., and 24.c.). This suggests the use of another -and probably softer and less durable- type of stone for these decorated parts.

As for the structural damages, it is known from geotechnical investigations in and around the church that the soil is quite weak, with some man-made fillings in some parts. Therefore, most of the cracks can be associated with soil settlements. The cracks on the “Walls” are further progressed with differential settlements between themselves which are lighter, and the heavy “Buttresses”. However, after one year of monitoring, it was observed that the cracks did not have a significant opening trend, apart from seasonal changes; and the soil settlements were accepted to have occurred during construction and therefore “not-active” any more.

Earthquakes, although with not very high peak around accelerations during the 14th and 15th centuries, and also explosions in the Civil War in Spain, can also be considered as possible damaging factors. These are known to be harmful mostly for the main and the lateral façades; however their impacts on the rest of the building can not be avoided.

D) Conclusion on the apse

During the visual inspection of the apse of Santa Maria del Pi Church, in the historic center of Barcelona, it was seen that the apse represented a similar crack pattern on all its façades. This was mostly comprised of “V” shaped cracks on top of the windows and also some additional ones under
them. In many parts, cracks were accompanied with repointings and sometimes with partial reconstructions that were easily distinguishable. The reason for this kind of cracks was thought to be the differential settlements between the “heavy” buttresses and much “lighter” walls.

Non-structural damages were limited with vegetation and loss of mortar generally. However, it was noted that these two phenomena accelerated each other’s progress.

7. EVALUATION OF COLLAPSE MECHANISM

7.1. Definition of the Demand Curve

Based on previous work developed by (Cuzzila, 2008) and the results of the geotechnical survey it was possible to identify that the foundation soil of Santa Maria del Pi has the worst conditions in terms of dynamic response and mechanical properties. Hereby for the construction of the demand curve, the worst possibilities in terms of soil conditions were taken into account, at the end the type 1 elastic response spectrum with soil conditions E was used to represent the amplification of the seismic waves in the ground and the parameter shown in (Table 1) were calculated according to the Eurcode 8.

<table>
<thead>
<tr>
<th>S</th>
<th>TB</th>
<th>TC</th>
<th>TD</th>
<th>PGA [m/sec²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.40</td>
<td>0.15</td>
<td>0.50</td>
<td>2.00</td>
<td>0.3924</td>
</tr>
</tbody>
</table>

Table 1- Parameter used in the calculation of the Eurocode 8 response spectrum

The spectrum obtained is presented in Figure 27. For compare the demand curve with the capacity curve it is necessary to present the demand curve in terms of spectrum displacements against spectrum accelerations. By having the spectrum acceleration and the period, it is possible to find the pseudo displacement, and then the demand curve in terms of spectral acceleration and pseudo displacement (Figure 28).

![Figure 27- Calculated elastic response spectrum](image)
The demand curve presented in Figure 28 will be used to compare the capacity curve and to determine whether the structure has an elastic demand or a plastic demand and how much is the safety factor for each particular collapse mechanism.

7.2. Most Critical Collapse Mechanism

According to (Cuzzilla, 2008), there are twelve different possible collapse mechanisms, which are most probable to occur during a seismic event; the results obtained by (Cuzzilla, 2008) are presented in Table 2. Within this twelve mechanisms, there were selected the most weak mechanism in terms of safety factors; this is the lowest safety factors. The selected collapse mechanisms are the following:

- The 3rd: Overturning of the upper part of the main façade without taking into account the rose window.
- The 4th: Overturning of the upper part of the main façade taking into account the rose windows.
- The 5th: Overturning of the central part of the main façade.
Apart from the above mentioned collapse mechanisms, the safety factor of them is higher than 3. Following the results obtained by (Cuzzilla, 2008), the most critical mechanism were identified by an extensive visual inspection in the site and they show that there are several crack openings that help to develop the three most probable collapse mechanisms. Hereby, these mechanisms were analyzed following the procedures described in Cuzzilla’s work which are the limit analysis of the local parts of the structure and the definition of the capacity of each local part. After this, the capacity curve for each mechanism was compared with the demand curve obtained from the Eurocode 8 (described in the previous section) and finally the Fajfar method was applied to qualify the damage that may experience the local part of the structure when is subjected to the Spectrum Code earthquake. The results are presented in the following subsections.

<table>
<thead>
<tr>
<th>mechanism</th>
<th>typology</th>
<th>NCSE02 - Fajfar</th>
<th>EC8 - Fajfar</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>total tower</td>
<td>7.14</td>
<td>D0</td>
</tr>
<tr>
<td>2</td>
<td>upper tower</td>
<td>13.29</td>
<td>D0</td>
</tr>
<tr>
<td>3</td>
<td>upper facade (no r.w.)</td>
<td>2.94</td>
<td>D1</td>
</tr>
<tr>
<td>4</td>
<td>upper facade (with r.w.)</td>
<td>7.22</td>
<td>D2</td>
</tr>
<tr>
<td>5</td>
<td>whole façade</td>
<td>2.8</td>
<td>D1</td>
</tr>
<tr>
<td>6</td>
<td>transept</td>
<td>4.82</td>
<td>D0</td>
</tr>
<tr>
<td>7</td>
<td>in-plane upper façade</td>
<td>14.42</td>
<td>D0</td>
</tr>
<tr>
<td>8</td>
<td>in-plane whole façade</td>
<td>14.89</td>
<td>D0</td>
</tr>
<tr>
<td>9</td>
<td>apse 1</td>
<td>3.88</td>
<td>D0</td>
</tr>
<tr>
<td>10</td>
<td>apse 2</td>
<td>4.28</td>
<td>D0</td>
</tr>
<tr>
<td>11</td>
<td>apse 3</td>
<td>3.95</td>
<td>D0</td>
</tr>
<tr>
<td>12</td>
<td>apse 4</td>
<td>5.97</td>
<td>D0</td>
</tr>
</tbody>
</table>

Table 2- Results of different collapse mechanism analyzed by Cuzzilla, 2008. (Cuzzilla,2008)

<table>
<thead>
<tr>
<th>label of damages</th>
<th>spectral displacements</th>
<th>damages description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D0</td>
<td>Sd ≤ 0.7 Sdy</td>
<td>no damage</td>
</tr>
<tr>
<td>D1</td>
<td>0.7 Sdy ≤ Sd ≤ Sdy</td>
<td>lightly damage</td>
</tr>
<tr>
<td>D2</td>
<td>Sdy ≤ Sd ≤ 0.25 Sdu</td>
<td>moderate damage</td>
</tr>
<tr>
<td>D3</td>
<td>0.25 Sdu ≤ Sd ≤ 0.5 Sdu</td>
<td>extensive and severe</td>
</tr>
<tr>
<td>D4</td>
<td>Sd &gt; 0.5 Sdu</td>
<td>completely damage or</td>
</tr>
</tbody>
</table>

Table 3- Damage qualification according to Fajfar method
7.2.1. Overturning of the upper part of the façade (not including the rose window)

For this particular mechanism only the overturning of the upper part of the façade was taken into account, but the rose window was discarded; there are several cracks that confirm the probability of this mechanism and they were already presented in the previous section. The area taken into account to perform the calculation is presented in Figure 29.

By applying the principle of the virtual work to the mechanism with the forces and distances presented in Table 4, the obtained force multiplier is $\alpha=0.134$.
Table 4 - Parameters used in calculation of the virtual work principle

The displacement was varied until reach the collapse of the local part of the structure and the capacity obtained is presented in Figure 30.

![Capacity curve](image)

**Figure 30 - Capacity curve in terms of force multiplier**

In order to compare the capacity curve in the same terms as the demand curve, the force multiplier was converted to spectral acceleration and the results are presented in Figure 31.

![Spectral capacity curve](image)

**Figure 31 - Capacity curve in terms of acceleration**

After this, the capacity and demand curves were compared in terms of the same parameters and the results are presented in Figure 32.
By intercepting the Ts curve and the capacity curve the obtained demand displacement of the seismic zone is 0.074m. According to the Italian recommendations the capacity displacement of the collapse mechanism is the 40% of the ultimate displacement; hereby the capacity displacement of the structure is 0.150m. Making the relation between the capacity displacement and the demand displacement the calculated safety factor is 2.03 for this particular collapse mechanism.

For the level of damage calculation, the Farjar method was applied, obtaining the results shown in Figure 33. This level of damage indicates that for the particular supposed earthquake, the damage that can be caused is moderate.

<table>
<thead>
<tr>
<th>s_d</th>
<th>s_dy</th>
<th>0.7 s_dy</th>
<th>s_dy</th>
<th>0.25 s_du</th>
<th>0.5 s_du</th>
<th>s_du</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.042 m</td>
<td></td>
<td>0.060 m</td>
<td>0.094 m</td>
<td>0.188 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.375 m</td>
</tr>
</tbody>
</table>

Figure 33 - Level of damage for the analyzed mechanism
7.2.2. Overturning of the upper part of the façade (including the rose window)

This particular collapse mechanism involves the upper part of the main façade taking into account the collapse of the rose window; the region which is influenced by this possible collapse is presented with yellow hatch (Figure 34).

Figure 34 - Collapse mechanism of the upper part of the façade (including the rose window)

To apply the principle of the virtual work, the following parameters were taken into account and the obtained horizontal force multiplier was $\alpha=0.046$.

<table>
<thead>
<tr>
<th>$P$ (ton)</th>
<th>$h_i$ (m)</th>
<th>$P$ (ton)</th>
<th>$d_i$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>450.23</td>
<td>8.20</td>
<td>450.23</td>
<td>0.38</td>
</tr>
</tbody>
</table>
Table 5 - Parameters used in calculation of the virtual work principle

The virtual displacement was varied and several force multipliers were obtained for each displacement until reaching the collapse of the mechanism and the results are presented in Figure 35.

![Capacity curve](image)

Figure 35 - Capacity curve in terms of force multiplier

In order to compare the results of the demand curve with the capacity curve, the capacity curve was converted in terms of acceleration and displacements and the results are presented in Figure 36.

![Spectral Capacity Curve](image)

Figure 36 - Capacity curve in terms of acceleration

The capacity curve and the demand curve were compared to find the safety factor of the collapse mechanism in terms of displacements and the results are presented in Figure 37.
The demand displacement of the structure is found by the intersection of the Ts line and the capacity curve, whereby the found displacement was 0.150m. The capacity displacement for this particular seismic zone according with the Italian code is the 40% of the ultimate displacement; hereby the found demand displacement is 0.134m. Comparing both of the obtained results it is possible to calculate the safety factor in terms of displacement and the result shows that the demand displacement of the structure is 1.12 times the capacity displacement of the structure, which is a rather low value.

For the calculation of level of damage Fajfar method was applied and the results are presented in Table 6.

<table>
<thead>
<tr>
<th>Sd</th>
<th>0.134 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7 Sdy</td>
<td>0.042 m</td>
</tr>
<tr>
<td>Sdy</td>
<td>0.060 m</td>
</tr>
<tr>
<td>0.25 Sdu</td>
<td>0.094 m</td>
</tr>
<tr>
<td>0.5 Sdu</td>
<td>0.188 m</td>
</tr>
<tr>
<td>Sdu</td>
<td>0.375 m</td>
</tr>
</tbody>
</table>

Table 6 - Level of damage for the analyzed mechanism

This level of damage indicates that for the demand seismic event the selected collapse mechanism may suffer from extensive damage to severe damage.
7.2.3. Overturning of the central part of the façade

For this particular mechanism the central part of the main façade was taken into account including the two small buttresses that are represented by areas A3 and A4. This particular mechanism was validated by the visual inspections which are showing large longitudinal repointed cracks along the height, just in the division of the towers and the main façade; the areas involved in this mechanism are presented in Figure 38.

![Figure 38 - Collapse mechanism of the central part of the façade](image)

In order to apply the virtual work principle the results shown in Table 7 were used to perform the calculations. These are representing the forces representing the different areas, and so after performing the calculations the force multiplier was $\alpha=0.060$. 
Table 7 - Parameters used in calculation of the virtual work principle

<table>
<thead>
<tr>
<th>P (ton)</th>
<th>h (m)</th>
<th>P (ton)</th>
<th>d (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>450.23</td>
<td>21.73</td>
<td>450.23</td>
<td>1.03</td>
</tr>
<tr>
<td>492.66</td>
<td>7.47</td>
<td>492.66</td>
<td>0.75</td>
</tr>
<tr>
<td>66.59</td>
<td>12.37</td>
<td>66.59</td>
<td>0.75</td>
</tr>
<tr>
<td>61.41</td>
<td>12.17</td>
<td>61.41</td>
<td>0.75</td>
</tr>
</tbody>
</table>

In order to obtain the capacity curve, several displacements were applied and the force multiplier was obtained for each one of them and the results are presented in the Figure 39.

Figure 39 - Capacity curve in terms of force multiplier

For make the comparison between the capacity and the demand curve, the capacity curve should be presented in terms of accelerations and displacements, hereby the acceleration for each particular force multiplier was obtained and the results were plotted in the Figure 40.

Figure 40 - Capacity curve in terms of acceleration
In Figure 41 are presented the results obtained by comparing the capacity and the demand curves for the particular collapse mechanism.

![Demand Vs Capacity](image)

**Figure 41 - Comparison between capacity and demand curves**

The demand displacement of the structure’s mechanism is obtained by finding the intersection between the Ts curve and the capacity curve and the obtained result was 0.163m. The capacity displacement of this particular mechanism is the 40% of the ultimate displacement and the obtained displacement was 0.337m. The safety factor in terms of displacement is 2.07, which indicates that for the particular seismic event, the structure may resist a displacement of 2.07 times the demanded one.

The results for calculate the structure’s level damage according to Fajfar method are presented in Table 8. For this particular case the results show that the structure may experienced a moderate level of damage when is submitted to the code’s seismic event.

<table>
<thead>
<tr>
<th>$s_d$</th>
<th>0.163 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.7 s_dy$</td>
<td>0.095 m</td>
</tr>
<tr>
<td>$s_dy$</td>
<td>0.135 m</td>
</tr>
<tr>
<td>$0.25 s_{du}$</td>
<td>0.211 m</td>
</tr>
<tr>
<td>$0.5 s_{du}$</td>
<td>0.422 m</td>
</tr>
<tr>
<td>$s_{du}$</td>
<td>0.843 m</td>
</tr>
</tbody>
</table>

**Table 8 - Level of damage for the analyzed mechanism**
7.2.4. **Overturning of the central part of the façade** (including the actual leaning levels)

This mechanism is involving the same regions as the previous one; the only difference is that the actual leaning of the façade was taking into account. The actual leaning of the façade was taken from (Vendrell, 2007) geometrical survey, which is showing a 0.30m of out of plumbs in the top of the thicker part of the façade. The scheme of the actual leaning is presented in Figure 42 as well as the region involved in the mechanism.

![Figure 42 - Collapse mechanism of the central part of the façade taking into account the actual leaning levels](image)

The values for apply the virtual work principle are presented in Table 9. The main difference with the previous mechanism’s values is that the vertical distance $h_i$ and the horizontal distance are modified by the out of plumb of the gravity center for each block. The calculated force multiplier for this particular case was $\alpha = 0.044$. 
The displacement was varied and the force multiplier was calculated for each one of them, and the results are plotted on Figure 43.

![Capacity curve in terms of force multiplier](image)

In order to compare the capacity curve of the mechanism with the demand curve of the seismic event, it is necessary to plot the results in terms of acceleration and displacement; hereby they are presented in Figure 44.

![Spectral Capacity Curve](image)
The comparison of the capacity curve and the demand curve is presented in Figure 45, which allows calculating the safety factor in terms of displacement.

![Figure 45 - Comparison between capacity and demand curves](image)

The demand displacement was obtained by the intersection between the capacity curve and the Ts curve, and the result was 0.161m. The capacity displacement was obtained as the 40% of the ultimate displacement hereby the result was 0.248m. By computing the safety factor of the results, there was possible to establish that the demand displacement is 1.54 times the capacity displacement.

For the calculation of the level of damage according to Fajfar theory, the data used is presented in Table 10.

|   |  
|---|---|
| $S_d$ | 0.161 m |
| $0.7 S_{dy}$ | 0.069 m |
| $S_{dy}$ | 0.099 m |
| $0.25 S_{du}$ | 0.155 m |
| $0.5 S_{du}$ | 0.310 m |
| $S_{du}$ | 0.620 m |

Table 10 - Level of damage for the analyzed mechanism

The obtained result indicates that for the demand of the seismic event, the level of damage that may have the structure for this particular collapse mechanism is from extensive level to sever level.
7.2.5. Overturning of the central part of the façade (including the horizontal thrust of the vaults in the longitudinal direction)

7.2.5.1. Calculation of the total weight of the vaults

In order to calculate the horizontal thrust of the vaults, both the longitudinal and transversal directions of the church were taken into account and the followed process was to calculate the volume of each vault material and then it was multiplied by its specific weight; the detailed process is described in the Annex GG.

The final results are presented here and they were obtained just by the summation of all the total volumes multiplied for their density values:

- Stone’s vaults: $8,1239 \times 2200 \text{ kg/m}^3 = 215872.58 \text{ kg}$
- Good quality filling: $1.8647 \times 2200 \text{ kg/m}^3 = 4102.428 \text{ kg}$
- Second filling with a worst quality: $255,30880 \times 1054.176 \text{ kg/m}^3 = 269140.46 \text{ kg}$

At the end the final weight of the total upper part of the vault is:

$$W_{\text{total upper part of the vault}} = 215872.58 \text{ kg} + 4102.428 \text{ kg} + 269140.46 \text{ kg} = 489115.468 \text{ kg} = 4796.58 \text{ KN} = 481.387 \text{ ton}$$

This will be distributed for each column in equal part. That means that will be equal to $\frac{1}{4}$ of the total value.

$$W_i = \frac{1}{4} \text{ of the total vault} = 122278.876 \text{ kg} = 1199.14 \text{ KN} = 120.34 \text{ ton}$$

In order to obtain the horizontal thrust line by graphic statics’ method, the force was drawn in Autocad using the following scale: $1 \text{ m} = 20 \text{ ton}$

7.2.5.2. Estimation of the horizontal thrust line

In order to study the horizontal forces due to the vaults, has been carry out the limit analysis. In the longitudinal direction, the only section that could suffer this force is the façade. As made in evidence in the picture, in the rest of the naves, all the vaults are balancing each other.

To solve this problem has been drawn one possible thrust line passing through the high quality stones, in the upper part of the vault, and across the first filling material in the lower part. Taking into account the thrust line and the weight coming from the vault, has been possible obtain the horizontal force. In the Auto CAD drawing, using the $W_i=120.34 \text{ ton}$ (weight of $\frac{1}{4}$ of the total vault) and the thrust line, has...
been possible to obtain the value of the horizontal force, which resulted equal to 0.7m. To represent these values in the drawing, the scale chosen was 1m=20 ton, the final value was 14ton.

![Figure 46 - Plan and Section views of the horizontal thrust lines in the longitudinal direction](image)

The horizontal force coming from the vault, acting in the longitudinal direction is:

\[ F_h = 14 \text{ton} = 139 \text{KN} \]

This value, if compared with the average of the forces acting in this kind of structures, is a very small one. That is the reason why the horizontal force does not influence the safety factor studied by the kinematic limit analysis, and why it can be ignored. This sentence will be widely proved in the following chapters.

Concerning the horizontal force acting along the transversal direction of the church, is evident that the dimensions and the weight of the buttresses are enough big to guarantee that any damage could occurs in this direction. At list can guarantee that any damage could be related with this kind of forces. Moreover, the thrust line and the weight coming form the adjacent vaults are balancing each other as shown in the following image.
7.2.5.3. *Analysis of the mechanism*

This mechanism is the same than the one analyzed in section 7.2.3, where the obtained force multiplier was 0.060. The results on (Table 11) show that for this particular case the force multiplier is equal to 0.058 and the difference between them is only 3.3%, this is showing that the horizontal thrust effect can be neglected.

Figure 47 - Plan and Section views of the horizontal thrust lines in the transversal direction
### 7.3. Conclusions about the collapse mechanism analysis

By changing the soil conditions from type B to type E, the obtained safety factors are lower than those obtained by (Cuzzilla, 2008). The results obtained by the analysis presented before are more reliable because they are representing with a better level of approximation the found soil conditions which are reported in the Geotechnical investigation.

It was demonstrated that is not worthwhile to take into account the horizontal thrust of the vault in the longitudinal direction because it only represents a difference of the 3% between the obtained results of the same mechanism without taken it into account this force, and regarding that its quantification is a time consuming task it can be neglected.

The obtained safety factors are the following:

- The 3rd: Overturning of the upper part of the main façade without taking into account the rose window: 2.03.
- The 4th: Overturning of the upper part of the main façade taking into account the rose windows: 1.12.
- The 5th: Overturning of the central part of the main façade: 2.07.
- The 5th: Overturning of the central part of the main façade taking into account the actual leaning level: 1.54.

This shows that the mechanism which has more probabilities to occur is the one that is including the upper part of the façade taking into account the rose window collapse and the possible level of damage that may occur in case of the occurrence of a seismic event is from extensive to sever, hereby this mechanism needs some measures regarding the improvement of its behavior when it is submitted to horizontal loads.

It is important to notice the difference between safety factor regarding the collapse of the central part of the façade, if the façade was plumb the safety factor would be 2.07 but because the façade presents an actual level of leaning the safety factor is 1.54, which is the 75% of the results obtained.
without taking into account the out of plumb level. Hereby it is very important to take into account the real geometry for further calculations.

The overturning of the central part of the façade should be also carefully analyzed in order to find any possible retrofitting measure regarding its stability when it is subjected to an horizontal force such like an earthquake, because with the eventual presence of this phenomenon it could suffered from extensive to severe damages.

7.4. Seismic improvement of the Main Façade

One of the weakest collapse mechanisms of the structure is the out of plane collapse of the main facade. The overturning of this macro element has already been analysed, and the calculated safety factor is equal to 1.54.

The previously described failure can be prevented by improving the connection between the main facade and the lateral wall. One suitable solution could be the tying of the facade to the lateral wall by means of two steel ties at both sides of the facade. Each tie can be anchored at the lateral vaults, by drilling the vaults at a height equal to the one third of their rise, which is a distance of 19.18 m from the lowest part of the facade. The total length of the ties can be estimated up to a distance of three spans of the lateral vault.

However, the calculation of the safety factor after the intervention is necessary, in order to evaluate the contribution of the ties to the improvement of the seismic behaviour. Moreover, the capacity of the ties needs to be determined for designing purposes.

The overturning of the tied facade is presented at the following picture:

Figure 48 – Overturnig of the tied facade
The position of the formed hinge is considered to coincide with the lowest part of the rose window, at a distance of 3.14m from the tie. The wall is separated into two blocks. The lower block is symbolized as block 1 and the upper block is symbolized as block 2. Block 1 has 16.04m height, while its width is changing. The width is equal to 1,42m at its base and, at a certain height, is decreasing up to 0.75m which continues until the top of the block. Block 2 has a height of 3,14m and a constant width of 0.75m.

The acting point of all forces is considered to be the barycenter of each block. A rotation of $\psi=1$ degree is given to block 1. Block 2 is rotating with an angle $\phi$. The barycenter of each block is rotating too, which can be analysed at a horizontal and vertical displacement, $\delta x$ and $\delta y$ respectively.

The values of the rotation angle $\phi$ and the displacements of each barycenter are given at the following table:

<table>
<thead>
<tr>
<th>$\psi(\degree)$</th>
<th>$\phi(\degree)$</th>
<th>$\delta 1 x (m)$</th>
<th>$\delta 1 y (m)$</th>
<th>$\delta 2 x (m)$</th>
<th>$\delta 2 y (m)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>5,110</td>
<td>0,157</td>
<td>0,013</td>
<td>0,157</td>
<td>0,047</td>
</tr>
</tbody>
</table>

Table 12 – Calculations performed for rotation angles and displacements

The weight of each block can be calculated by each area of the facade multiplied by the respective thickness and the specific weight of masonry, which is equal to $\gamma=21,57\text{KN/m}^3$.

The results are presented at the next table:

<table>
<thead>
<tr>
<th>area 1($m^2$)</th>
<th>area 2($m^2$)</th>
<th>area 3($m^2$)</th>
<th>$\gamma (\text{KN/m}^3)$</th>
<th>P1(KN)</th>
<th>P2(KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>153,100</td>
<td>41,420</td>
<td>26,060</td>
<td>21,570</td>
<td>2679,717</td>
<td>210,793</td>
</tr>
</tbody>
</table>

Table 13 – Calculations performed to obtain weights

Each area corresponds to the respective part of the facade, as shown below:

Figure 49 – Corresponding areas
The weights have been calculated by the following formulas:

\[ P_1 = (\text{area 1} \times 1.42 + \text{area 2} \times 0.75) \times 21.57 \times 0.5 \]

\[ P_2 = \text{area 3} \times 0.75 \times 21.57 \times 0.5 \]

The equivalent earthquake forces that cause the instability of the wall are assumed to be equal to the weight of each block multiplied by a multiplier \( \lambda \):

\[ F_1 = P_1 \times \lambda \]

\[ F_2 = P_2 \times \lambda \]

According to the principle of virtual work, the work of the external forces applied to the structure is equal to the work of the internal forces. This can be shown by the following expression:

\[ P_1 \delta_{1y} + P_2 \delta_{2y} = F_1 \delta_{1x} + F_2 \delta_{2x} \Rightarrow \]

\[ P_1 \delta_{1y} + P_2 \delta_{2y} = \lambda (P_1 \delta_{1x} + P_2 \delta_{2x}) \]

The value of \( \lambda \) is shown at the table below:

<table>
<thead>
<tr>
<th>P1</th>
<th>P2</th>
<th>( \delta_{1x}(m) )</th>
<th>( \delta_{1y}(m) )</th>
<th>( \delta_{2x}(m) )</th>
<th>( \delta_{2y}(m) )</th>
<th>( \lambda )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2679,717</td>
<td>210.793</td>
<td>0,157</td>
<td>0,013</td>
<td>0,157</td>
<td>0,047</td>
<td>0,098</td>
</tr>
</tbody>
</table>

Table 14 – Calculations performed to obtain \( \lambda \)

The value of the axial force of each tie can be computed by the equilibrium equation of the moment at the rotation point of the first block:

\[ F_1 \times 9,02 + F_2 \times 17.404 - P_1 \times 0,74 - P_2 \times 1,05 - RT \times 19.184 \Rightarrow \]

\[ RT = 27,137 \text{ KN.} \]

The value of the seismic coefficient for the specific mechanism is considered to be rather small. As a result of this, the value of each tie that is needed to prevent this collapse mechanism is also low. Based on these facts it can be concluded that the requirement of placing steel ties is not integral, considering all the historical implications, the workman hours, the complexity of the intervention and the uncertainties involved on it.
8. MONITORING AND NON / MINOR DESTRUCTIVE TESTING

Monitoring and non/minor-destructive testing (NDT/MDT) are two integral parts in investigating a historic building and/or trying to assess its structural safety. The necessity of establishing the building integrity or the load carrying capacity of a masonry building arises for several reasons including:

(i) assessment of the safety coefficient of the structure (before or after an earthquake, or following natural/accidental events like hurricanes, fire, etc.);

(ii) change of use or extension of the building;

(iii) assessment of the effectiveness of repair techniques applied to structures or materials; and

(iv) long-term monitoring of material and structural performance (Binda, 2001, p. 30).

Nevertheless, the design of the repair interventions must be approached only after a deep knowledge of the material and construction techniques have been obtained; and this goal is the motivation of most monitoring and NDT applications (Binda, ?, p. 1).

Monitoring consists of periodic or continuous surveys carried out by means of instruments preferably connected to a data acquisition unit to record the parameters such as: deformations, out-of-plumbness (tilting), strains, movements in joints or cracks, temperature/relative humidity/wind variations, settlement of foundations, variations in ground water tables, etc. (Croci, 1998, p. 65).

The types of NDT available at present are mainly based on the detection of the physical properties of a structure; and most frequently applied in situ tests can be listed as sonic/ultrasonic, radar and thermography tests which provide qualitative results. These need to be interpreted in a careful manner by specialists in accordance with the person (architect/engineer) responsible for the design of interventions. However, this task is mostly -if not always- a challenging one due to the great number of uncertainties related to historic masonries.

Amongst the in situ mechanical tests, flat-jack tests may be accepted as the most applicable to historic masonry structures. The single and double flat-jack tests give local measurements of stress and stress-strain relationship respectively; however they are minor destructive (MDT) and thus not applicable in certain cases (such as in the presence of precious surface decorations, etc.).

Laboratory tests also provide valuable information on the consistency, structure/composition and behavior (resistance, deformability) of building materials. However, these can also be regarded as minor destructive tests, requiring some amount of sampling; thus not always applicable.

8.1. Previous Studies

Certain monitoring and NDT/MDT were carried out in Santa Maria del Pi Church recently, in order to understand better the structure in terms of construction techniques, building materials, deformations, etc.
8.1.1. Monitoring of Cracks

To determine the movement of the most important cracks, three displacement transducers were installed, measuring the movements in three directions (longitudinal, transversal and perpendicular to the wall). These were placed:

1. On the left side of the main façade, installed at the level of the choir near the rose window;
2. At the junction of the main and lateral façades (the “Epistola” side);
3. On the left side of the nave’s roof (the “Evangeli” side) (Figure 50).

![Figure 50 - One of the transducers placed on the roof](image)

The temperature changes were also monitored (on the roof, at the “Epistola” side), to distinguish the cyclical movements from the actual ones; and it was observed after a monitoring period of 6 months (June-December 2007) that the movements were mostly caused by daily and seasonal temperature effects (Figure. 51, 52).
8.1.2. Direct and Indirect Inspections

8.1.2.1. The Subsoil and the Foundations

In order to establish the characteristics of the subsoil and the foundations, following works were carried out:

- Two penetrometric drillings at the portal of Ave Maria and the main façade, with a depth of 20 m;
- A rotational drilling with a diamond core, at the junction of the main and lateral façades (at the *Epistola* side), with a depth of 25 m;
- An archaeological excavation to understand the geometry, materials and construction of the foundations, at the portal of Ave Maria, up to a depth of 2 m;
- Investigation with Georadar to detect areas/structures of different densities, at the perimeter of the church, in different profiles, until a depth of 3-4 m (three different antennas were utilized with frequencies of 200, 400 and 900 MHz) (Figure. 53, 54);

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The results obtained through subsoil and foundation analysis are explained in detail in Chapter 5.1.
- Inspection with the Nakamura\textsuperscript{8} technique, to understand the nature of the subsoil in and near surroundings of the church (Figure. 55).

8.1.2.2. The Roof and the Vaults

To determine the structure of the roof, both direct and indirect inspections were carried out, such as\textsuperscript{9}:

- Excavation of a number of “inspection holes (cales)” to understand the system of impermeabilization, and the filling materials in the roof of the chapels and the nave;

- Investigation with Georadar in longitudinal profiles, over the chapels and the perimeter of the nave, including the apse (Figure. 56, 57);

\textsuperscript{8} The concept of the Nakamura technique is to measure the environmental vibrations in the surface, and thus to obtain the fundamental frequency of the site.

\textsuperscript{9} The structure of the roof and the vaults, including the information obtained through GPR are explained in Chapter 5.3.2.
8.1.3. Physicochemical Analysis

8.1.3.1. Characterization of Building Materials

For the characterization of materials, samples were taken from accessible points of the building and following analysis were carried out\textsuperscript{10}:

- Optical microscopy with various configurations;
- Electron microscopy and X-ray analysis to identify the trace elements;
- Infrared spectroscopy;
- X-ray diffraction.

8.1.3.2. Mechanical Properties and Durability of Materials

Since the properties of Montjuic Stone (which is the main building material used in the construction) is very well known, mechanical tests were kept at a minimum in order not to disturb the structure. However, the “hole drilling\textsuperscript{11}” technique was used to detect the local compression stress level on the right side of the main door, before dismantling the engraved Roman stone block, found during the archaeological excavation.

\textsuperscript{10} The characterization of materials is described in Chapter 4.
\textsuperscript{11} The technique is based on drilling a small cylindrical hole and measuring the deformations through a system of electrical currents, in which circuits are installed in a circular mechanism.
8.1.3.3. Thermal Behaviour of the Vaults

Understanding the thermal behaviour of the vaults was considered necessary for two reasons:

- To be able to approximate the dynamics of heat transmission through the vaults, in order to interpret the displacements of the cracks due to thermal expansion;

- To establish the safety factor of the nave due to the thermal inertia of the vaults and its heat transfer in both directions.

The analysis showed that, in each section, a similar pattern could be observed with slight variations; with the parts of the vaults that are facing SE being warmer than those facing NW, because of the interaction angle with the sun (Figure. 58). It was also concluded that the thermal inertia of the nave was quite low and thus affecting the environmental conditions within.

![Figure 58 - The thermal distribution of one of the vaults](image)

8.2. Proposals for Future Studies

Although an important amount of monitoring and NDT/MDT was carried out, as explained in the previous part, further studies can be implemented to understand better the Santa Maria del Pi Church, in order to have a better judgement during the design of interventions.
8.2.1. Analytical Architectural Survey

Analytical architectural survey of the building is necessary in terms of defining and mapping different characteristics of it in a systematic manner. To be able to do this, first the detailed surveys, showing all building materials in their exact geometry and position should be prepared. This could be done in the traditional way (hand-measurements); however, taking into account the size of the building, it may necessitate a huge amount of time. Other possible ways are using photogrametry or laser-scanning; but the latter is still very expensive, which leaves the first as a logical choice over the others.

After obtaining the detailed survey, following characteristics should be mapped on sets of drawings, by using different colours or hatches:

- Materials: Defining different materials (different stones/bricks/mortar types; metal or timber elements, etc.) helps to understand historic repairs and to determine intervention proposals;

- Chronology: Related with materials but also taking into account the different construction techniques when similar materials are used, the construction and repair chronologies can be established (some uncertainties will always remain).

- Damages: Structural and non-structural damages should be shown to understand better the causes that create them, and to design the interventions in a more appropriate way.

8.2.2. Archaeological Excavation

It is advisable to carry out an archaeological excavation in order to understand the depth, construction typology and materials of the foundations. This was done, up to 2 m in front of the main façade; however the final depth could not be reached, due to inadequate working conditions and to presence of public installations.

Necessary permits should be requested from the municipality, explaining the need and importance of such work; and arrangements should be made for installations. A possible location for the excavation could be around the portal of the lateral (“Evangeli”) façade, where the spacious Plaça de Josep Oriol is located.

In order to prevent the instability of the excavation a carefully excavation design should be performed by the geotechnical engineer with the help of the archeologist in order to be able to make an excavation depth enough to find the real depth of the foundation.

8.2.3. Monitoring

Monitoring should be expanded to a higher number of cracks, distributed evenly throughout the structure and it should at least continue for a period of four years. This would provide the necessary information to obtain reliable correlations between cyclical effects (such as temperature) and crack
movement and thus to eliminate them. Additional monitoring instruments could be installed in the following locations:

- On the main façade: As explained in further chapters, certain interventions are proposed for the cracks over the rose window (that are possibly associated with historic settlements); and those between the main façade and the two towers on each side (which may create dangerous collapse mechanisms) (See 9.2.4, 9.3.1 and 9.3.2) (Figure 59). However, it is important to monitor these cracks to understand both, if their movement is still active and also to evaluate the effectiveness of the interventions.

![Figure 59 - Main façade; possible locations for crack monitoring](image)

Cracks under the rose window can be accepted as characteristic features of churches with big rose windows, which occur because of the formation of a relieving arch and leaving these parts unloaded. These cracks are most probably non-structural and may not require monitoring.

- On the apse: Cracks over the upper windows, that may also be consequences of settlements; the two diagonal cracks on the first wall section after the “Evangeli” lateral wall; the

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12 Together with crack movements, environmental factors such as temperature, humidity, wind, sunlight, etc. should also be monitored.
13 Since it wasn’t possible to carry out an inspection in the inside of the church due to the great height, poor lighting conditions and scaffoldings/net, detailed proposals couldn’t be devised for this part. However, certain cracks were observed, mainly on the chapels’ vaults and a few on the nave’s walls. Still, decisions on these should be given upon extensive inspections.

Another part that couldn’t be inspected was the “Epistola” façade, which requires a similar attention as the interior. And since the most dangerous crack on the “Evangeli” façade, that separates the wall itself from the adjacent tower (and consequently the main façade) is already being monitored, no additional monitoring was proposed for this part.
movements around the Presbitery portal; and those on the annex building which may be pointing out to a still continuing settlement (Figure 60).

- On the roof: The continuous crack on the “Epistola” side, whose propagation is limited by the presence of neighbouring buildings on this side.

Still, since the instruments, their installation and the evaluation of monitoring data may be quite expensive, “telltales” or “plaster” may be used as a first attempt. More precise equipment may then be installed on the cracks where the initial monitoring means have been disturbed (broken/cracked).

The tilting trend of façades should also be monitored with inclinometers, to capture active soil settlements -if any. This should mainly be concerned on the front and the “Evangeli” façades, where higher levels of out-of-plumbness are measured during the architectural survey.

Another possibility for monitoring could be the dynamic one. However, since Barcelona is not a highly seismic region and the occurrence of a low magnitude earthquake is quite uncommon, this was regarded as unfeasible.

8.2.4. Soil Monitoring

To find out the main causes of soil settlement, it is necessary to know if there is any presence of low rigidity soils or water sensitive soils. Also it is important to verify the position of the building and its proximity with later heavy constructions. After this understanding it is possible to design a monitoring system that may allow to determine whether if the settlement is active or if it was active in the past but now it is stable; usually this activity takes place over a period of time involving mostly years rather than days.

Various methods of instrumentation are available ranging from traditional ‘manual’ techniques to electronic ‘automated’ systems, but it is necessary to define monitoring intervals, measuring accuracy,
allowed limits and consequent interventions. Also it is necessary to define which soil phenomena will be monitored in order to choose the right equipment to measure any soil movements, water pore pressure, applied loads, stress condition and crack opening.

Regarding Santa Maria del Pi Church, the geotechnical studies has already shown that the soil is composed mainly of a superficial man-made fill, a superficial layer of high deformable clay and an interior layer of marl detritus where silt is the main component, which is also highly deformable. The man-made fill layer has high fluidity, it is potentially collapsible and its nature is noncohesive. The superior clay layer is cohesive but it is also fluid, it has a medium plasticity, it is highly deformable and it has a high expansion potential. Finally the marl detritus layer has a lower expansion potential and the plasticity is medium but it also presents a high deformability property. In general, the geotechnical condition of the soil foundation of the church is mainly poor for the superficial layer and rather low for the middle layers, but the stress condition of the soil has suffered an incremental compact, consolidation and earning in bearing capacity processes. These may have occurred due to the different seismic events and water table oscillation cycles because they have fit all the soil particles into a better structural arrangement.

The water table level is -6.70m, and it could be very possible a fluctuation of this level. But with these results it is not possible to determine whether this phenomenon is still active or if it has stopped. Hereby a method which allows determining the activity of this phenomenon is required, and monitoring the soil conditions may present a good solution.

In order to define the next step in the geotechnical prospection, there are several conclusions in the geotechnical report which can give a clear target of the main activities to develop.

- There is a massive presence of sort of anomalies inside the superficial layers of the soil, mainly ruins and tombs, which can cause a substantial negative influence in the bearing capacity and compressibility ratio of the soil foundation. Hereby a more detailed information is necessary in order to identify the affection ratio in the bearing capacity.

- There is a lack of information of certain part of the soil foundation because the materials properties were obtained in only one bore hole which is not representative due to the heterogeneous conditions of the different soil layers and the large extension of area that is involved.

- The geotechnical prospection was done without knowing the total depth of the foundations because the stability of the foundation could be affected by the archeological excavation that was made, hereby the dimensions were extrapolated from the obtained results and the safety factor was impossible to calculate. Also it was not possible to know the transmitted loads from the walls of the perimeter and the buttresses to the foundation, which implies that the safety factor relating the acting loads from the foundation with the terrain bearing capacity is not possible to be calculated.
• The probability of the variation in the water table is high, hereby this circumstance should be considered as unfavorable because it may increase the soil's collapse potential, especially in the man-made fill and in sandy materials which present a Nspt<10. For the monitoring of this fact, a piezometer inside the S-1 was installed covering the 20 meters depth.

The influence of anomalies in the superficial layers is a very difficult task to know because this zone has a high archeological value and to develop a research regarding this purpose with the usual geotechnical approach many test should be performed and still the information obtained may not give a clear mechanical behavior of this soil layer because it is compounded by many different materials which are coming from several centuries ago and had been suffered many decomposing and deterioration processes.

One borehole soil prospection is recommended to cover the vast area that the church is occupying; its position in the plan is show in Figure 61. This kind of study is intended to have a better understanding of the poor region (the blue one) in terms of mechanical properties, and to correlate the results obtained in the previous borehole prospection, georadar results and geophysical prospection. It is recommended to carry out the same tests that were applied in the geotechnical prospection work previously carried out.

It should be useful to monitor the water level conditions in the main façade soil foundation especially in the adjacent building zone. There is one piezometer located in the red zone which is represented by S1 on Figure 61, hereby an additional piezometer is needed to be placed in the blue region, it can be similar to the previous one, and it can be placed in the same place as the recommended borehole inspection. It should be important to monitoring the Nspt parameter each three months in order to
compare with the reference level of 10 in order to give proper alarms of possible risk of soil’s collapse potential.

8.2.5. Qualitative Tests

In order to have a deeper understanding of the wall structure (its characteristics, hidden voids/cracks, etc.) qualitative tests such as sonic/ultrasonic or GPR can be implemented on certain locations, where the masonry seems most stressed/disturbed.

Since these tests are quite expensive in terms of both execution (necessity for scaffolding) and data interpretation -and require an important amount of time for the latter-, they should mainly be concentrated on the main façade and its connections with the towers and lateral façades, to understand the depth and extent of cracks seen from the exterior.

However, since historic masonries are highly heterogeneous, calibration and interpretation of the obtained data should be carefully carried out and cross-examined, in order not to create any confusions or unreliable test results.

Complementary or alternative to these tests (in case of economic or time limitations), borescopy/endoscopy can be carried out to directly verify the characteristics and anomalies. These should -if possible- be inserted into present voids/cracks, instead of opening new ones, to disturb the structure as less as possible. Flexible instruments, instead of rigid ones could be used to overcome the difficulties regarding possible irregularities of the masonry.

8.2.6. Further Studies

According to the results obtained through the above-mentioned monitoring and testing, further studies can be carried out, in case an active crack movement and/or settlement is detected. These should mainly concentrate on defining the causes of settlements which may be related with:

- The insufficient shear strength due to poor geotechnical soil conditions;

- The reduction in pore volume due the normal consolidation of the soil or any external factor that can cause the re-accommodation of the particles such as an earthquake or the variation of the water table; or,

- The reduction of grain volume due to creep effect which is caused by the long term loads applied to the soil.

Regarding this tasks, a more detailed geotechnical study should be developed in order to decide which are the main causes of the soil settlements. For the specific case of the poor bearing capacity, it is very important to know the level of stresses that the structure is transmitting to the foundations and then to the soil.

Although there are many possibilities to obtain quantitative data through tests, most of these are unfortunately inappropriate for historic buildings, requiring coring/drilling of samples, pulling out of
parts, etc. However, flatjack tests can provide information on local stress levels and stress-strain relationships (therefore on elastic modulus), with a limited extraction of bedding mortar which can easily be restored through repointing.

Hereby single flat jack tests can give a good approximation of the stress levels that are acting, and then it could be possible to calculate a safety factor to know by a quantitative way the intensity of this phenomenon. Thus, single flatjack tests can be used in the connections between the main façade and the towers, where continuous vertical cracks are observed, which most probably were caused by soil settlements. One flat-jack can be placed in the lower-right side of the main facade wall and the other one can be placed in the lower part of the left tower’s wall (Figure. 62).

![Figure 62- Positions of the single flat-jack tests](image)

The results obtained from flat-jack tests may then be used to calibrate structural models, which generally include a high number of uncertainties regarding the geometry, morphology, heterogeneity, etc. of structures.

9. INTERVENTION PROPOSALS

Santa Maria del Pi Church is an important architectural heritage building in Barcelona, which must be handed to future generations. The building has survived for centuries and resisted to actions of nature (long-term actions like soil settlements and weathering or sudden ones like earthquakes) and men (vandalism, explosions, inappropriate interventions) throughout its long history. These are evidenced by patinas, cracks, replaced materials, etc. on its surfaces. Although most of the causes for structural damages seem to be stabilized today, it is not known for certain how the structure will behave if a change in its conditions occur (soil/water table change); or a
seismic activity takes place. This is a main point of concern, since some collapse mechanisms present quite low safety factors.\textsuperscript{14}

Non-structural damages, such as vegetation, salt crystallization, inadequate re-pointing, etc., also present problems like attracting moisture (and thus providing possibilities for further damage) and exhibiting poor appearance.

Therefore, in order to safeguard the monument and to ensure its integrity, a number of measures should be taken. However, when defining/designing the intervention proposals, certain principles must be considered, such as:

- Minimum intervention; or best put “optimum” intervention;
- Reversibility, that is the possibility of returning to the condition, prior to the intervention;
- Respect for authentic materials;
- Retention of structural members and structural system; thus not creating drastic changes in stiffness which will cause problems in seismic activities;
- Maintenance; which is probably the most important principle in heritage preservation.

Thus, a number of interventions were devised, concentrating mainly on the front façade, which has several structural cracks, and which has almost the weakest mechanisms for collapse. Nonetheless, most of the proposals could be recognized as general recommendations for a complete restoration work.

Still, it must be noted that, it was unfortunately not possible to carry out a satisfactory inspection in the interior and the roof of the church, which are known to have some serious damages.

\section*{9.2. Non-structural Interventions}

\subsection*{9.2.1. Removal of Vegetation}

Plants throughout the façades present important problems, as already discussed in the previous chapters. These -although not structural yet- may become more dangerous if they are permitted to grow and distribute their roots inside the walls; and cause the mortar to lose its integrity, thus its capability of adhering the building blocks to each other. This may, in ultimate cases, even lead to structural damages, such as the appearing of cracks and/or large voids in the walls.

Therefore they need to be removed to avoid such problems. However, this should be carried out in a careful manner, in order not to destroy the joints or the stones, where the roots of the plants have already established. After removal, herbicides may be used to prevent future growth (the chemical formula should be chosen as appropriate with the characteristics of the building materials, to avoid an unwanted chemical reaction and deterioration).

\footnote{Roberto Cuzzilla, 2008}
9.2.2. Cleaning of Efflorescences

Efflorescences are mainly concentrated at the lower parts of the front façade and on the inner surfaces of the apse. These are crystallized, water-soluble salts that may have been carried to the structure in different ways (such as underground water, pollution, sea-spray, repairs with Portland cement, etc.) and result from the oscillations in temperature. Their extent on the façade is not drastic yet; however some parts of the apse walls exhibit distributed patterns. These can be removed with poultices prepared with de-ionized water or certain chemical compounds; or gels with special formulas to absorb the salts from the stones. After the execution, the surface should be cleaned again with de-ionized water and it must be made sure that all remains of poultice or gel are cleared to prevent further reactions. However, it is also important to understand the sources of salts, and try to eliminate them, before reaching the structure if possible. For this purpose, samples should be taken and analyzed in the laboratory and the nature and possible sources of salts should be determined.

9.2.3. Cleaning Birds’ Excrements

Birds’ excrements promote decay of the surface due to their composition and also create a bad appearance on the façades. They could be removed with similar techniques to those of efflorescences, with different chemical formulas. Measures should also be taken to prevent them roosting and nesting to avoid the reoccurrence of the problem as much as possible. This could be done by netting or spikes, at least in the decorated surfaces such as the main portal.

9.2.4. Repointing

Repointing is necessary in three different situations:

- Where the mortar in the joint is decayed/vanished: to prevent water ingress inside the structure;
- Where the mortar in the joint is inappropriate in (a) composition and/or (b) appearance: to (a) prevent adverse chemical reactions and/or to (b) re-establish the integrity of the surface; or,
- Where reinforcement of the joints is necessary: to ensure better behavior against seismic events (explained in detail in Chapter 9.3.2.2.).

Prior to repointing, the joints should be cleaned off remains of plaster to provide higher adherence during execution. The composition and grain size distribution of the re-pointing mortar should be based on the authentic lime mortar, defined with laboratory tests. Skilled craftsmen should be employed in order to avoid the necessity of an early “re-repointing” and to ensure good workmanship.
9.3. Structural Interventions

9.3.1. Injection

Voids in historic masonry walls may have been formed by mortar decaying and falling downwards and outwards through cracks or faulty joints; or alternatively by dissolving in acidic rain water and leaching away, forming micro-caverns in the structure. In the case of Santa Maria del Pi Church, it is highly probable that the walls bear voids to some extent. However, the size of voids is quite unpredictable by visual inspection. Still, simple tools, like tapping with a hammer and listening to the note; or more elaborate tools such as sonic/ultrasonic or radar tests can provide valuable information.

It is important to clean the voids and wet all surfaces, before inserting the mortar mix to ensure good adhesion. The mortar mix should be formulated specifically for the building; that is having similar characteristics to the original mortar. To increase penetration and flow properties, additives can be used; but the effects of modern compounds should be thoroughly investigated prior to application. Addition of pozzolanic material may also be necessary to ensure hardening in the absence of CO₂ of the air. The use of Portland cement should be strictly avoided, since it creates regions of high stiffness that are alien to the original structure; and may also cause efflorescence and/or cryptoflorescence.

The wall then can be consolidated by injection/grouting, that is filling the voids with the specified mortar mix. This can be done in a local manner -as injecting where the voids and cracks are located. Or in the case of a distributed void pattern (when detected by NDT), a regular grid of drillings can be made and a systematic injection applied through working first sideways and then upwards. In each case, the amount of grout inserted into the voids should be carefully recorded, as this provides a rough guide to the structural condition of the fabric.

9.3.2. Seismic Strengthening of the Main Façade

The slenderness of the main facade, the appearance of several open cracks and the weak connection between the facade and the lateral wall demanded a solution for strengthening against seismic actions. However, the choice of intervention should be adequate to the cultural value of the church, and avoid altering the external view, as well as the structural pattern of the building.

The employment of tie-bars that would connect the façade to the lateral walls, could not be justified by kinematic analysis, as the increase in the safety factor was not a satisfactory one (See Chapter 7.4.), and since this intervention necessitated drilling through the wall section and also some of the vaults. Nevertheless, a general description of this technique is presented here just to have an approximation of all the work that has to be carry out in order to demonstrate the effects that this intervention has.
9.3.2.1. Ties on the façade

A possible strengthening of the façade could contemplate the application of ties. In this section is reported the techniques required in order to evaluate the cost benefit analysis of the intervention. The main aim is to find the less invasive operation with the higher result in term of strengthening.

A) Ties position

According with the geometry of the façade and with the course of the vaults behind it, the best position found to put the tie is at 19.184m height from the basement, and at 8.2m from the symmetrical axes of the façade. This tie could run along the lateral wall, crossing the vaults in the filling zone as in presented in Figure 63 (good filling quality). This point is located where a possible thrust line is expected to pass. Anyway, the tie has to run along the lateral internal wall, in order to allow the fixing of the tie’s end on the wall.

![Figure 63- Position of tie within the vault](image)

This position is also suggested by the fact that the good state of art recommends applying ties in areas where the structure is able to counteract the compression force induced. That is one of the reasons why the tie could be applied along the lateral wall. Another reason is that the lower part of the vault is one of the most resistant. So that, if the plate head of the tie is oriented in order to allows the tensions, like is showing in Figure 64 (once they pass the wall façade) to be absorbed by the lower filling layer and by the good stones of the vault, that one will be one of the better solution. Below is reported in red a qualitative representation of the tie position.
As shown in the Figure 64, from the esthetic point of view the linear plate head of the tie could be one of the best typology. The other kind of plates, the elliptic one and the round one, are too massive and could disfigure the façade.

**B) Ties dimension**

Once known the entity of the forces that have to be retained, it is possible to define the dimension of the tie and its head. As a complementary step, the verification of the strength of the bar, and the wall resistance is necessary.

The pulling action of the tie, applied by its head on the masonry façade, creates a shear strength that has the same unitary value in both directions: parallel and orthogonal to the tie. This kind of actions generates traction solicitations, as represent in Figure 65.

One of the main problems that could occur is the collapse of the masonry due to traction. Is necessary that the force applied will be smaller than the one resisted. In this case, if the head tie is well located, as previously explained, the problem will not occur. By the way, the following formula could be checked in order to avoid this collapse:
\[ N = \sigma_0 (4S - 2s + 2L)S\sqrt{2} \]

N is the maximum value that could be applied to the masonry. The other values refer to the geometry properties of the head of the tie and are shown in Figure 66.

Figure 66 – Geometry properties of the head of the tie

Once designed the tie, will be necessary to assure that all the main façade will benefit by this strengthening. In order to obtain a homogeneous and global behavior of the bound part and the rest of the façade, it will be necessary to apply some M bars joint of FRP (methodology already explained in the chapter 9.3.2.2.). This kind of bars will connect the area surrounding the tie with the central part of the façade. To make efficacious this connection, is suggested to extend the bars versus the rose window for at least 1m from the end of the internal vault.

This method will be treated deeply in the specific chapter. In Figure 67 is reported a possible distribution of these bars. Surely, a mortar retrofitting of the studied area will increase the efficacy of the intervention.

Figure 67 – Distribution of the bars
C) Tie placing

To place the tie is necessary to drill the façade and the vaults. Considering the dimension of the forces that have to be retained, is not needed a big portion of the longitudinal wall to resist them. Indeed just a small section of the lateral wall will be enough to resist the instability forces acting in the façade. Therefore, will be enough to drill just the first vault, instead the first three, and anchor the tie on the lateral wall. The position suggested for this anchorage could be 15cm far from the pillar. Follow this proposal a better ratio between the damages caused to the structure and the retrofitting advantages, could be obtained. In this case, the drilling length required will be equal to the façade section plus the thickness of the vault plus the thickness of the column:

\[ L_{drilling} = 0.75m + 0.78m + 1.76m = 3.29m \]

D) Tie support

If the application of the tie is chosen, the last test that will have to be carried out is the check of the forces applied on the longitudinal wall. In this specific case, has to be taken into account its effect on the equilibrium moment in the column.

According with the trust line previously found, the forces coming from the vaults will be acting on the middle section of the column. The weight of the column will act on its barycenter and the tie force will be applied at 19.18m height.

To find the weight coming from the column itself, its area has been evaluated taking into account 0.95m of thickness. The specific weight used is the stone vaults one. So that, the final volume resulted equal to:

\[ V_{column} = 10.66 \times 0.95 = 10,127 m^3 \]

The final value of the column weight corresponds to:

\[ W_{column} = 10,127 \times 2200 = 22279.4 \text{ Kg} = 21.92 \text{ ton} \]

Even in this case the drawing scale has been taken 1m = 20ton.

At this point it has been possible to check the equilibrium acting in the pillar. As known, the half weight of the total vaults is acting on it because there are two vaults loading in the same pillar. With the knowledge of the dead load of the column, and the force due to the tie, it has been possible to check if the Stabilizing Moment is bigger than the Destabilizing Moment.

\[ M_{stab} - M_{destab} > 0 \]
\[ M_{\text{stab}} = \left( \frac{W_{\text{vault}}}{2} + W_{\text{pillar}} \right) \times S/2 = \left( \frac{481.38}{2} + 21.92 \right) \times 0.5/2 \text{ ton m} \]

\[ M_{\text{destab}} = F_{\text{tie}} \times H_{\text{tie}} = 1.522 \times 19.18 \text{ ton m} \]

\[ M_{\text{stab}} - M_{\text{destab}} = 65.6525 - 29.19 = 36.46 > 0 \]

It has been confirmed that this kind of tools doesn’t affect the safety of the lateral nave’s wall.

9.3.2.2. Crack-Stitching Technique

An appropriate solution, that would both strengthen the structure and not affect its appearance or cause unnecessary drillings could be the “stitching” of cracks by means of bars (retro-reinforcement); which at the same time decrease the possibility of development of new cracks. The bars, usually of a diameter from 6 to 10 mm and a length of at least 1 m are embedded into the horizontal joints of the masonry and the joints are repointed upon implementation.

The application method of these bars is due to technical recommendations reported by several companies. According to these recommendations, the application procedure is the following:
- The mortar in the joints is removed to a depth up to 25-35 mm, a width of 8 mm, and a length of 0.5 m from both sides of the crack. The removal of mortar can be executed by a diamond-headed drill or a wheel (Figure. 69). This step is actually quite important to avoid the creation of a weak area in the masonry;

![Figure 69 - Removal of mortar from both sides of the crack with a wheel](image)

- Any remains of mortar is removed with pressed air or water to avoid damaging the head of the drill (Figure. 70);

![Figure 70 - Cleaning the remains of mortar with pressed air or water](image)

- The “back” of the joint is filled with mortar (Fig. 71);

![Figure 71 - Filling the “back” of the joint with mortar](image)

- The bars are put and secured in the joint. If the crack is next to a corner (at a distance smaller than 500 mm) or an opening (at a distance smaller than 100 mm), the bar should be bended at the corner and anchored there. (Figure. 72);
Figure 72 - Putting and securing the bar in a horizontal joint

- Mortar is poured, so that the perimeter of the bar will be covered up to the external face. Then the extra mortar is removed and the joint surface is smoothed, as in a regular repointing work (Figure. 73).

Figure 73 - Finishing the joint surface for a smooth look

9.3.2.3. Application of Crack-Stitching on the Main Façade

Considering the fact that the collapse of the upper part of the facade is one of the weakest mechanisms (with a safety factor equal to 1.69, taking into account the whole facade), the placing of reinforcement bars for crack stitching can be justified.

The strengthening bars that will be used are FRP Mbars Joint with 5 mm diameter. They are made from carbon fibers, embedded in matrix epoxy. After the insertion, the bars will be covered with Albaria Struttura which is a kind of lime mortar with pozzolanic additives.

The open cracks generally appear at the upper part of the facade, above the rose window. They are referred as F-2c, F-9c and F-1c (Figure. 74):
Taking into account the total height of the façade over the rose window (~ 4.96 m on the left and ~ 5.18 m on the right), the total courses of stones (18), and considering an average of 2 mm of joint thickness, the height of each stone block is calculated approximately as 26 cm\(^{15}\).

The bars could be placed every two courses, which would leave them ~ 54 cm (2 stones + 1 joint) apart from each other. This distance is beyond the usual application practices; however, placing the bars in every course was considered as quite too much.

Thus, the number of bars needed for each crack is (Figure 75):

- F-1c: The crack runs for 12 courses of stone. By placing the bars every 2 courses, 6 bars are needed to "stitch" it. Since this crack is very close to a corner (less than 50 cm in most of its length), the bars should be bent and anchored at the part between the facade and the tower (Figure 76).

\(^{15}\) This preliminary and non-accurate calculation should be substituted with the real values, after the analytical architectural survey has been prepared.
Figure 75 - Bars that will be used to “stitch” the open cracks (in blue)

- F-9c: The crack runs for 15 courses of stone. By placing the bars every 2 courses, 7 bars are needed.

- F-2c: The crack runs for 13 courses of stone. By placing the bars every 2 courses, 6 bars are needed. This crack runs through 2 stone blocks; and it may also be necessary to replace them. However, this decision should be left to the execution phase.
10. COST ESTIMATION

In order to estimate the cost of the previously suggested interventions the required quantities of materials and manwork need to be taken into account.

The strengthening of the upper part of the facade is based on removing of mortar at a depth up to 2cm, placing the bar and covering the bar with “Albaria struttura”. For this specific intervention the number of reinforcement bars and their length is calculated. However, since the price of the bars has not been specified, these values can be used for future evaluation of the cost. The calculation results are shown at the following table:

<table>
<thead>
<tr>
<th>FRP Bars</th>
<th>Φ=5mm</th>
<th>N bars</th>
<th>Total length (m)</th>
<th>Price per m</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>crack 1c</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>crack 2c</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>crack 9c</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>2</td>
<td>14</td>
</tr>
<tr>
<td>total</td>
<td>19</td>
<td>19</td>
<td>2</td>
<td>38</td>
<td></td>
</tr>
</tbody>
</table>

Table 15 – Demanded amount and length of bars

The time needed for removing and replacing the mortar of each joint has been assumed to be equal to one hour and a half per each mortar line, and the value of the demanded for the application manwork equal to 20€ per hour. The price of the required mortar has been considered equal to 2,34 €/kg. The cost of the mortar and the manwork is estimated as following:

<table>
<thead>
<tr>
<th>removed mortar</th>
<th>required mortar</th>
<th>price per</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total length (m)</td>
<td>Total area m³</td>
<td>Kg</td>
</tr>
<tr>
<td>6</td>
<td>0,0024</td>
<td>1700</td>
</tr>
<tr>
<td>6</td>
<td>0,0024</td>
<td>1700</td>
</tr>
<tr>
<td>7</td>
<td>0,0076</td>
<td>11,1384</td>
</tr>
<tr>
<td>19</td>
<td>0,0076</td>
<td>11,1384</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>manwork</th>
<th>Total length (m)</th>
<th>cost €/h</th>
<th>(h/m)</th>
<th>€/m</th>
<th>price (€)</th>
<th>total price €</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>20</td>
<td>1,5</td>
<td>30</td>
<td>180</td>
<td>570</td>
<td>980,2328</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>180</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td>210</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td>570</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 16 - Cost estimation of removed mortar and manwork
The cost estimation of the recommended inspection techniques, such as monitoring of the cracks on the facade, inclinometers for the soil settlements and tiltometers for the tilting of the facade, cannot be fulfilled totally, due to lack of information about their costs. However, an attempt to evaluate the cost of some of the proposals is shown below:

<table>
<thead>
<tr>
<th>Name of Test</th>
<th>N of tests</th>
<th>Cost for test</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat jack (1)</td>
<td>2</td>
<td>750</td>
<td>1500</td>
</tr>
<tr>
<td>Radar Inspection</td>
<td>1</td>
<td>-</td>
<td>25000</td>
</tr>
<tr>
<td>Inclinometers</td>
<td>2</td>
<td>500</td>
<td>1000</td>
</tr>
</tbody>
</table>

Table 17 – Cost estimation of NDT

The cost for scaffolding for the front façade can be estimated as:

<table>
<thead>
<tr>
<th>Area of necessary scaffolding</th>
<th>Cost for m²/month</th>
<th>Time</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>27 m²</td>
<td>10</td>
<td>24 months</td>
<td>6480</td>
</tr>
</tbody>
</table>

Table 18 – Cost estimation of scaffolding

The total cost that corresponds to the previously estimated values is the following:

<table>
<thead>
<tr>
<th>Cost of mortar</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost of FRP bars</td>
<td>38</td>
</tr>
<tr>
<td>Cost of manwork</td>
<td>980</td>
</tr>
<tr>
<td>Cost of flatjack</td>
<td>1500</td>
</tr>
<tr>
<td>Cost of radar inspection</td>
<td>25000</td>
</tr>
<tr>
<td>Cost of inclinometers</td>
<td>1000</td>
</tr>
<tr>
<td>Cost of scaffolding</td>
<td>6480</td>
</tr>
</tbody>
</table>

Table 19 – Total cost of the required mortar, manwork and flatjack tests
11. REFERENCES

12. ANNEX 1 (photographic survey “Main” Façade)

12.1. LOCALIZATION OF PHOTOS

Figure 1.a. - Plan of structural reported damages
Figure 2.a. - Plan of non-structural reported damages
12.2. MAPPING

Figure 3.a. - Mapping of structural damages

Figure 4.a. - Mapping non-structural damages
12.3. STRUCTURAL DAMAGE

The pictures below were taken from different angles and position on the main façade (Figure 1 and 2). Each crack can be identified with a typology on the mapping (Figure 3 and 4).

Picture 5.a. – Crack (F-1c) Step-like crack associated with the connection between the east tower and the main façade

Picture 6.a. – Crack (F-2c) Cracks appearing due to soil settlements
Picture 7.a. – Crack (F-3c) Repointed step cracks near the tower connection

Picture 8.a. – Crack (F-4c) Repointed compatibility crack radiating from the rose window
Picture 9.a. – Crack (F-5c) Compatibility cracks radiating from the rose window. They have been repointed but the crack on the left appears to be reopening

Picture 10.a. – Crack (F-6c) New crack at mortar joints between portal and façade wall. Crack continues around the portal connection
Picture 11.a. – Crack (F-6c) Decaying mortar joint at the beginning or end of the portal connection crack. Mortar crumbles at the touch. Crack is approximately 5mm wide.

Picture 12.a. – Crack (F-6c) Vertical crack through stone approximately 5mm wide.

Picture 13.a. – Crack (F-7c) Vertical cracks of varying widths in portal stone. Most likely non-structural, but could be evidence of separation due to bending.
Picture 14.a. – Crack (F-8c) Repointed step crack

Picture 15.a. – Crack (F-9c) Repointed crack apparently stabilized
12.4. NON-STRUCTURAL DAMAGE

The pictures below were taken from different angles and position on the main façade. (Figure 12)
Each damage can be identified with a typology on the mapping. (Figure 14)

Picture 16.a. – Crack (F-10c) Repointed crack between the tower and façade

Picture 17.a. – Crack (F-11v) Plantlife next to the west tower, rooted in mortar joints
Picture 18.a. – Crack (F-12v) Plantlife next to east tower, rooted in mortar joints

Picture 19.a. – Crack (F-13 Plantlife growing between gaps in shingles v)
Picture 20.a. – Crack (F-14pl) Restitution of plaster by means of silicic cement mortar

Picture 21.a. – Crack (F-15v) Moss growing in vacant mortar joints
Picture 22.a. – Crack (F-16v) Moss growing in vacant mortar joints

Picture 23.a. – Crack (F-17v) Black patina shows evidence of moisture
Picture 24.a. – Crack (F-18p)

Picture 25.a. – Crack (F-18p) White stains show evidence of efflorescence
Picture 26.a. – Crack (F-19r) Black patina at edge of rose window shows evidence of moisture saturation

Picture 27.a. – Crack (F-20d) Large void in stone
Picture 28.a. – Crack (F-21pl) Bad restoration by means of plaster
13. ANNEX 2 (photographic survey “Main” Façade)

13.1. LOCALIZATION OF PHOTOS AND MAPPING

Figure 1.b. – “Evangeli” façade of the Santa Maria del Pi Church, showing the photo positions
13.2. PHOTOS

Picture 2.b. – Connection of main façade with lateral wall. Presence of vertical cracks

Picture 4.b. – Connection of main façade with lateral wall. Presence of stones of different size.
Picture 5.b. – Connection of main façade with lateral wall. Presence of stones with different size.

Picture 6.b. – Masonry in bad condition.
Picture 7.b. – Construction Time - history. Later addition (thickness of 20cm and length of 17m)

Picture 8.b. – Stones cutted to make pass some kind of light cable
Picture 9.b. – Soil slope. Probable soil settlements

Picture 10.b. and 11.b. – Construction time – history. Later addition (thickness 20 cm and length 17m)
Picture 12.b. – Filling in windows. Loss of mortar in the joints

Picture 13.b. – Construction time – history. Different stone’s color between the basement and the upper wall
Picture 14.b. – Construction time – history. Different stone’s color between the lateral door and the wall

Picture 15.b. – Previous interventions
Picture 16.b. – Reduced thickness of the buttresses on the lower part corresponding to the lateral columns of the door

Picture 17.b. – Lateral door. Different inclination of the right and left arch
Picture 18.b. – Reduced thickness of the buttresses in the lower part corresponding to the lateral jambs of the door

Picture 19.b. – Probable leaning of the left buttress. Probable soil settlement
Picture 20.b. – Different inclination of right and left arch of the lateral door

Picture 21.b. – Construction time – history. Different stone’s color of the basement and the upper wall
Picture 22.b. – Crack starting from the lower part of the window until reaching the floor level. Probable different movement between the apse and the lateral wall.
14. ANNEX 3 (photographic survey “Apse” Façade)

14.1. LOCALIZATION OF PHOTOS

Figure 1.c. - Plan of the Santa Maria del Pi Church, showing the photo angles
14.2. MAPPING

Figure 2.c. - Damage maps of the apse façades of the Santa Maria del Pi Church
14.3. PHOTOS

Picture 3.c. - General view of the apse

Picture 4.c. - General view of “Wall 1”
Picture 5.c. - Lower portion of “Wall 1”

Picture 6.c. - Middle portion of “Wall 1”
Picture 7.c. - Upper portion of “Wall 1”

Picture 8.c. - The small “bridge” on “Wall 1”
Picture 9.c. - Cracks over the window on “Wall 1”

Picture 10.c. - General view of “Wall 2”
Picture 11.c. - Lower portion of “Wall 2” showing the annex

Picture 12.c. - Middle portion of “Wall 2”
Picture 13.c. - Upper portion of “Wall 2”

Picture 14.c. - Construction of the annex on “Wall 2”
Picture 15.c. - General view of “Wall 3”

Picture 16.c. - Lower portion of “Wall 3” showing the annex
Picture 17.c. - Lower portion of "Wall 3"

Picture 18.c. - Middle portion of "Wall 3"
Picture 19.c. - Upper portion of “Wall 3”

Picture 20.c. - General view of “Wall 4”
Picture 21.c. - Lower portion of “Wall 4” showing the annex

Picture 22.c. - Middle portion of “Wall 4”
Picture 23.c. - Upper portion of “Wall 4”

Picture 24.c. - Cracks and repointings over the window on “Wall 4”
Picture 25.c. - Upper portion of “Wall 5” (Lower parts can’t be seen because of the annex)

Picture 26.c. - The annex building in front of “Wall 5”
Picture 27.c. - “Buttress A”

Picture 28.c. - “Buttress B” - Side a
Picture 29.c. - “Buttress B” - Side b

Picture 30.c. - “Buttress C” - Side a
Picture 31.c. - “Buttress C” - Side b

Picture 32.c. - “Buttress D” - Side a
Picture 33.c. - "Buttress D" - Side b

Picture 34.c. - "Buttress E" - Side a
Picture 35.c. - “Buttress F”

Picture 36.c. - Neighbour building
Picture 37.c., 38.c. - Damages observed on the neighbour building
Picture 39.c., 40.c. - Damages observed on the neighbour building
15. ANNEX 4 (Study of the gothic vaults)

The Church of Santa Maria del Pi is an important example of the Catalan Gothic architecture. In this chapter is studied its gothic vaults. In order to find their weight, has been necessary to perform a 3D model using AutoCAD. The result, thanks to the small approximations done, has a good accuracy. Using the section provided from the previous works, has been possible to estimate the vaults’ dimensions.

The model was divided in 3 different parts, according with the materials present.

- Stone’s vaults: the best quality of the Montjuic stone was used here in order to guaranty the goodness of its structural behavior. The thickness used in this calculation, according with the precedent drawn section, has as an average of 50cm.

- First Filling: this kind of material was modeled with a height equal to 1/3 of the total vaults rise. That means equal to 5,25m.

- Second Filling: using this notion has been defined the upper part of the vault. In this case, the complex composition made by filling of bad quality, by brick’s floors and by void space, was modeled as a unique volume. The mass’ value has been estimate as the average of the different materials present.

The result obtained is the following:

![Figure 1.d. Longitudinal section](image1.png) ![Figure 2.d. Transversal section](image2.png)
The final 3D model obtained is the following:

![3D model](image)

**Figure 3.d. 3d model**

The different materials modeled are represented in three different colors in this Figure, each color corresponds to the following parts: Stone's vaults in blue, First Filling in green and Second Filling in red.

*Vault made by Montjouic stones:*

![Vault](image)

**Figure 4.d. Vault**
For this structural part of the vault, the stones’ properties are the following:

- Mass: 2200 Kg/m3
- Young Modulus: 8000MPa
- Compression resistance: 8MPa

Using AutoCad tools has been possible to evaluate the exact volume:

![AutoCad screenshot]

Thanks to that has been possible, for this part of the structure, to evaluate the total weight, which is equal to:

$$W_{\text{vault}} = 98.1239 \text{m}^3 \times 2200 \text{ kg/m}^3 = 215872.58 \text{ kg} = 2116.98 \text{ KN} = 212.46 \text{ton}$$

**Filling in the 1/3 of the vault:**
This kind of filling material is present only in the first 1/3 of the vault rise. Its properties are the following:

- Mass: 2200 Kg/m3;
- Young Modulus: 400MPa;
- Compression resistance: 4MPa.

Also in this case the values used are in accordance with the “S.M. del Pi” relation. By AutoCad has been obtained the required values:

The total weight of this part of the structure is equal to:

\[ W_{\text{filling}} = 1.8647 \text{m}^3 \times 2200 \text{ kg/m}^3 = 4102.428 \text{ kg} = 40.23 \text{KN} = 4.03 \text{ ton} \]

*Upper filling and roof composition*
To solve this complex case, at first has been found total volume using AutoCad tool. As a second step has been estimated the average mass of the different material presents, according with their percentage in the volume. The total volume is the following:

\[
W_{\text{filling}+\text{roof}} = 255,308,000 \text{m}^3 \times 1054.176 \text{ kg/m}^3 = 269140.46 \text{ kg} = 2639.36 \text{ KN} = 264.88 \text{ton}
\]

As already explained, to find the weight coming from the upper part of the vaults, has been necessary to make an average of the specific weights of the different materials present. In order to obtain this value, has been necessary to find the percentage of each material present in the total volume (see picture below). Once did that, has been possible to obtain the final specific weight average. The correct shape of the upper composition is reported in the following drawing, where is possible identify three different layers:

Figure 6.d. Roof composition
• **Roof percentage**

For the upper part, corresponding with the roof one. Here has been found its exact volume. Knowing the specific properties of the bricks used, has been possible to find in which percentage this part is influencing the total weight:

![Diagram of roof structure]

The area estimate is $15,867.4$ m$^2$ which correspond to the **25.9%** of the total one.

The property used for this material is the following:

- Mass: $1835.488$ Kg/m$^3$

**Second filling percentage**

This part corresponds to the filling, which is loading on the previous one. This material has worst properties than the other.

![Diagram of filling structure]

Its area corresponds to **51.6%** of the total one. The values used to define its properties are the following:

- Mass: $1121.68$ Kg/m$^3$
**Voids percentage**

After it have been valuated the percentage of the voids, which are present in this part of the roof.

The voids’ area is equal to 13.69m² which corresponds to 22.5% of the total volume.

The values used to define the material properties were extracted from the following Italian masonry classification OPCM 3431 2005:

<table>
<thead>
<tr>
<th>Tipologia di muratura</th>
<th>$f_{ck}$ (N/cm²)</th>
<th>$f_{ct}$ (N/cm²)</th>
<th>$E$ (N/mm²)</th>
<th>$G$ (N/mm²)</th>
<th>$w$ (KN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Muratura in pietra disgregata (ciotelli, pietre esotiche e irregolari)</td>
<td>90</td>
<td>2,0</td>
<td>690</td>
<td>113</td>
<td>19</td>
</tr>
<tr>
<td>Muratura a cenci chiusi, con paramento di lamina spessore e nucleo interno</td>
<td>110</td>
<td>3,5</td>
<td>1020</td>
<td>170</td>
<td>20</td>
</tr>
<tr>
<td>Muratura in pietra a spaccio con buona tenuta</td>
<td>150</td>
<td>5,6</td>
<td>1500</td>
<td>250</td>
<td>21</td>
</tr>
<tr>
<td>Muratura a cenci di pietra tenera (tufo, calcarenite, ecc.)</td>
<td>80</td>
<td>3,8</td>
<td>900</td>
<td>150</td>
<td>16</td>
</tr>
<tr>
<td>Muratura a blocchi lapidei squadriti</td>
<td>400</td>
<td>7,8</td>
<td>2410</td>
<td>400</td>
<td>22</td>
</tr>
<tr>
<td>Muratura in mattoni pensi e malta di calce</td>
<td>180</td>
<td>6,0</td>
<td>1100</td>
<td>300</td>
<td>18</td>
</tr>
<tr>
<td>Muratura in mattoni componi con malta cementizia (cso. doppi UNI)</td>
<td>380</td>
<td>6,0</td>
<td>2800</td>
<td>560</td>
<td>15</td>
</tr>
<tr>
<td>Muratura in blocchi laterizi forniti (perc. foratura &lt; 45%)</td>
<td>460</td>
<td>10,0</td>
<td>3400</td>
<td>682</td>
<td>12</td>
</tr>
<tr>
<td>Muratura in blocchi laterizi forniti, con grani verticali a serco (perc. foratura &lt; 45%)</td>
<td>400</td>
<td>13,0</td>
<td>3100</td>
<td>530</td>
<td>11</td>
</tr>
<tr>
<td>Muratura in blocchi di calciostruzzo (perc. foratura tra 45% a 65%)</td>
<td>150</td>
<td>9,5</td>
<td>2200</td>
<td>440</td>
<td>12</td>
</tr>
<tr>
<td>Muratura in blocchi di calciostruzzo sempieri</td>
<td>300</td>
<td>12,5</td>
<td>2800</td>
<td>560</td>
<td>14</td>
</tr>
</tbody>
</table>

$f_{ck} = $ resistenza media a compressione della muratura  
$f_{ct} = $ resistenza media a taglio della muratura  
$E = $ modulo di elasticità normale  
$G = $ modulo di elasticità tangenziale  
$w = $ peso specifico medio della muratura

The average value that has been use to obtain the final weight of the vault is the following:

\[ W \times m^{3} = 1835,48 \times 0,259 + 1121,68 \times 0,516 = 1054,1762 \text{ KNm}^{3} = 10.33 \text{ KNm}^{3} = 1.03 \text{ ton m}^{3} \]
16. ANNEX 5 (Intervention Mapping)

Figure 1.e. Intervention Mapping