



ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS



Master's Thesis

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Historical Case Study: The Frari's Basilica in Venice

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ABSTRACT

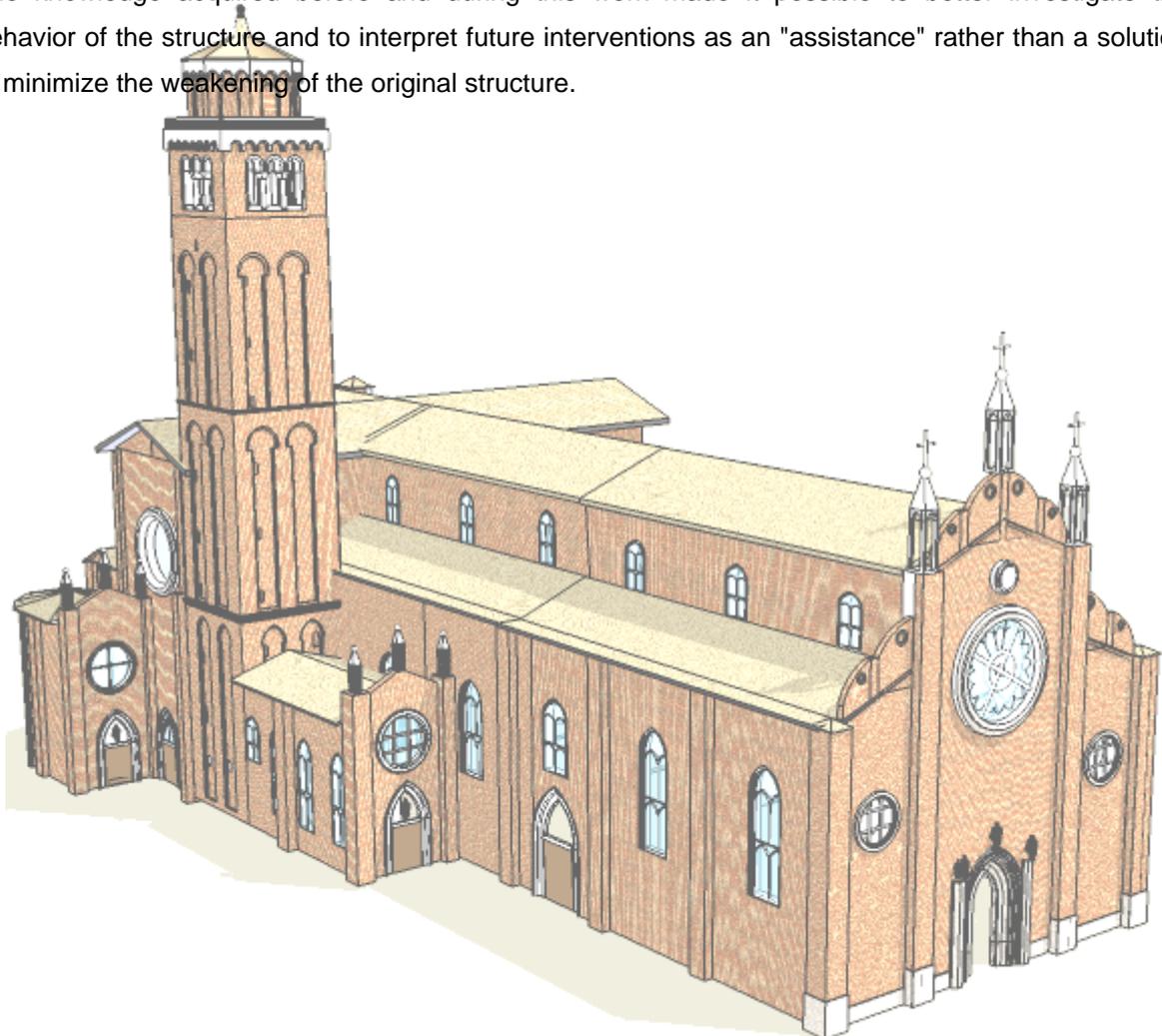
This dissertation presents the main features of the "S. Maria Gloriosa dei Frari" Basilica which was built in Venice between the first half of the XIII and the second half of the XV century.

The structural integrity of this ecclesiastical complex was immediately compromised and suffered from differential settlements due to the improper bearing capacity of the soil. Attempts at strengthening the structure over the course of the last five centuries proved to be laborious and for the most part ineffective.

This thesis will focus on the study of the various structural consolidations, monitoring systems and tests performed on the site in recent years past interventions. Included in this work will be a partial F.E.M model of the Basilica which serves to better interpret the results of both past and more recent experimental investigations and monitoring up to 2004.

A careful cognitive process made it possible to interpret, in detail, the issues related to subsidence of the soil upon which the foundation had been built which ultimately led to a significant crack pattern within the inner walls of the Basilica. Also included in this analysis is the study of the past interventions at solidifying the structure, specifically the bell tower, which was constructed in a different period of time and using different methodologies.

The knowledge acquired before and during this work made it possible to better investigate the behavior of the structure and to interpret future interventions as an "assistance" rather than a solution to minimize the weakening of the original structure.



SOMMARIO

Il presente elaborato ha il proposito di argomentare le principali caratteristiche della Basilica di Santa Maria Gloriosa dei Frari, costruita a Venezia tra la prima metà del XIII secolo e la seconda metà del XV secolo. L'integrità strutturale di questo complesso ecclesiastico, composto dalla Basilica, da un campanile e da tre principali cappelle, è stata fin dall'inizio compromessa dalla scarsa capacità portante del suolo veneziano; ciò ha portato a cedimenti differenziali tra le principali strutture sopra citate. I molti tentativi mirati al rinforzo e al consolidamento delle strutture durante questi ultimi cinque secoli hanno dimostrato essere molto laboriosi ma purtroppo allo stesso tempo inefficaci.

Il presente elaborato ha il compito di capire e descrivere l'evoluzione degli studi che stanno alla base dei numerosi interventi strutturali di restauro e rinforzo, nonché di rappresentare la concezione e l'installazione del sistema di monitoraggio e i risultati dei numerosi test eseguiti in cantiere nel corso della più recente fase d'intervento. Incluso nel presente documento vi è anche la discussione dei principali risultati emersi da un modello parziale agli elementi finiti della Basilica che è servito a meglio comprendere e valutare le conseguenze di entrambi gli interventi strutturali, passati e recenti.

Un accurato processo di conoscenza ha reso possibile interpretare in dettaglio i problemi legati ai cedimenti differenziali del terreno sottostante, sul quale le fondazioni sono state costruite; tali cedimenti hanno recentemente causato la formazione di preoccupanti fenditure nei muri interni alla Basilica.

Le informazioni acquisite prima e durante la stesura del presente elaborato hanno reso possibile un miglior apprendimento del reale comportamento delle strutture e hanno lo scopo di dare un contributo alla soluzione del problema dell'indebolimento delle strutture del complesso ecclesiastico.



1. PROJECT BRIEF

The "Basilica of Santa Maria Gloriosa dei Frari" is located in centre of Venice.

Its construction history has been a troubled one since the beginning and few documents to evaluate the exact year of its building process are available. Nevertheless it is ascertained that the ecclesiastic complex was built by the will of the Franciscan monks which is a mendicant brotherhood based on the rules left by St. Francis of Assisi.

Since its beginning the Basilica suffered from differential settlements which presented an interesting crack pattern between the bell tower and the transept area of the church. Since the various constructions were built in the weak soil of the Venetian lagoon there is no difficulty in understanding why such massive complex suffered such a fate. Furthermore the structures (namely the Basilica and the bell tower), primary built separately to allow differential settlements, have been lately mistakenly been seen as jointed together (locally) highlighting even more the crack outline especially in the interface wall that separates the transept from the bell-tower (SW wall).

The survey and the monitoring of the Basilica have been carried out specifically in this portion of the complex with focus on the interactions between the *bell tower-Basilica* and, most of all, *bell tower-soil* which is the main cause of the instability.

The passing of centuries brought forth technological advances which naturally resulted in more advanced intervention techniques. These technological advances have been used to evaluate and attempt to rectify the problems of the church which occurred over the course of the centuries.

Present scenario:

From the last diagnosis report (year 2004), compared with the reports from the beginning of the XX century, it is evident that the configuration of the cracks seems to have stabilized. In more modern times the second phase of the works, which started in 1990, is nearing its end and a new phase, which includes the removal of the provisional static protection applied in the previous phase and the periodic monitoring of the movements of the church, is about to commence.

The aim of this work is to illustrate the techniques and the construction phases of the Basilica's complex, describe the methodologies used during the past centuries' interventions and consciously read and elaborate the results of the experimental tests and the monitoring system in order to propose a different investigation plan to be accompanied by the actual one. This accomplishment includes the evaluation of the results of the finite element model compared with the in-situ investigations.

2. HISTORICAL SURVEY

Historians belief places the first settlement of Friars Minor (*Frati Minori*) in Venice during the third decade of the thirteenth century.

While historical documentation confirms the existence of a deed of donation dated July 7th 1234 from *Doge* Jacopo Tiepolo to the Order of Dominicans of an area near *Santa Maria Formosa*, the donating deed of an area in *St. Stin* to the Friars Minor remains unfound. Its absence is most likely a direct result of a fire at the friary which occurred in 1369.

Therefore, the settlement of the Franciscans in Venice, thanks to the donation of the *Doge* Tiepolo, took place in 1331 in the territory of the former lake called Badoer (Latin: *Lacus Badovarius*). This first church was to be of modest size, masonry and wooden ceiling, similar to *St. Maria of Padova*, erected in the same years and now incorporated in the *Basilica del Santo*.

Definitely better documented is the existence of the second church of *Frari*, as highlighted by many historians. The parchment that certifies the laying of the foundation stone and the assignment of the name of "S. Maria Gloriosa" is preserved in the Antoniana Library in Padua. This document states the presence, during the ceremony, of the apostolic Cardinal Ottaviano Ubaldini and three bishops, including the one "from the Castle" (that is the bishop from Venice), on April 28th, 1250.

However, the building proved to be too narrow for the huge crowd of faithful and as such a new, larger construction was completed by 1338.

Most experts agree that the opening of this new construction site occurred around 1330, as confirmed by a document dated 13th July 1330 in which the "Signoria" granted to the Franciscans a new portion of land to build the new ecclesiastic building. The orientation of the new structure is opposite respect to the one of the church of the XIII century probably because of the need to use the old church during the construction of the new one. Besides, the choice of the new orientation would have given the opportunity to turn the front of the Basilica towards the center of the city. As with other Gothic churches experts agree that the work would have started from the apse, presumably to carry on the construction with the transept, starting from the northern side, contiguous to the friary's buildings.

After a stop in the years 1370-80 due to limited economic resources, work resumed in the last decade of the fourteenth century. The church received many donations for its construction; among the many the most noteworthy is that of four columns and walls by the Grandenigo family in 1391, showing that in this period the body of the church had advanced until the actual front Façade.

After another setback of the work, during the last years of the fourteenth century and the first two decades of the fifteenth century, the friars took over the construction of aisles. The previous small church was taken down and the full perimeter of the new construction was laid. Evidence supporting this theory shows an existing document dated 1420 which contains the dimensions of the proposed church which actually correspond to the dimensions of the current structure (Figure 2-1).

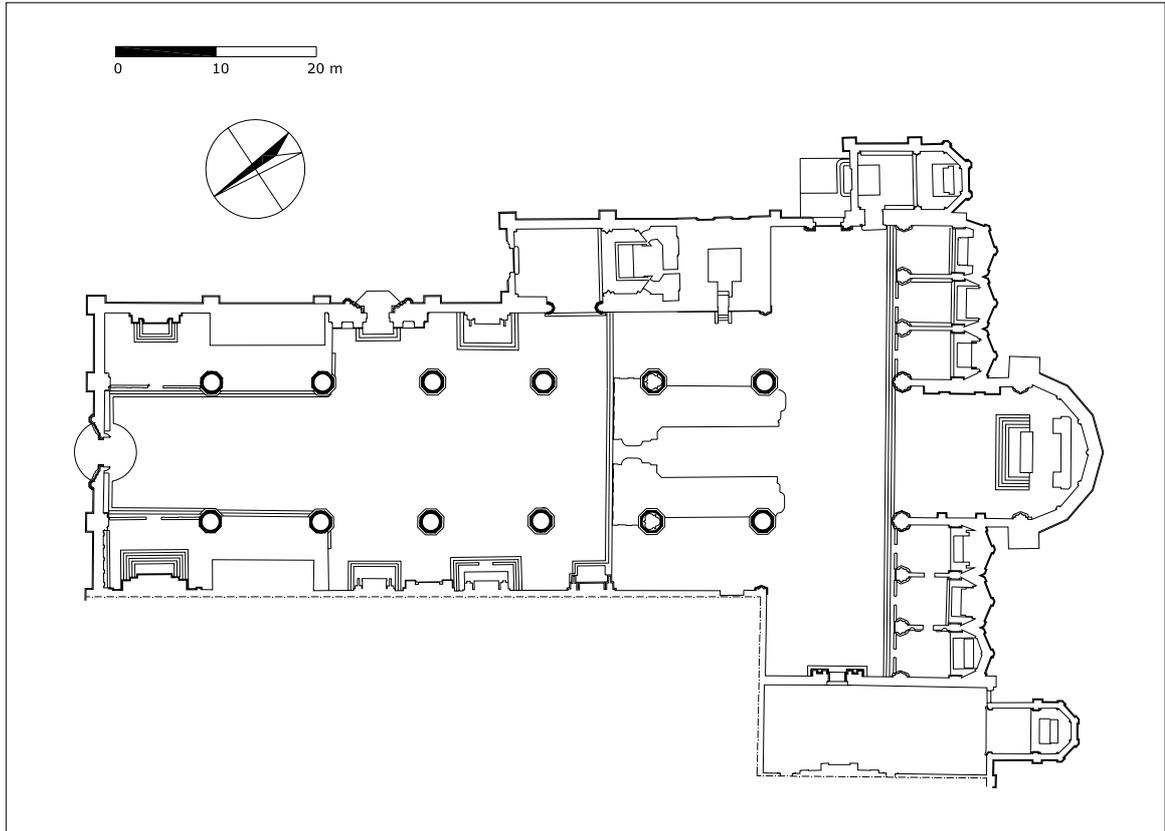


Figure 2-1: Plan view of the Basilica

It can be inferred that superstructure of the aisles was completed around mid fifteenth century. The final construction phase also includes the vaults of the last part of the aisles; later, namely fifteenth century, are instead the double cross vaults of the fourteenth century's transept, which overlaps the existing structure with many irregularities, for example, the large stone pillars are not big enough to support the vault ridges that do not have sufficient bearing support.

It can further be inferred that superstructure of the aisles was completed around mid fifteenth century. During the same period also the final vaults of the aisles have been constructed. In 1417 work on the Corner's Chapel began thanks to the last will and testament of Federico Corner on 15th March 1378; the chapel is leant to the southern wall of the transept and is closed by a pentagonal apse unlike the apses of the church which have an equal number of sides (four or six) (Figure 2-2).

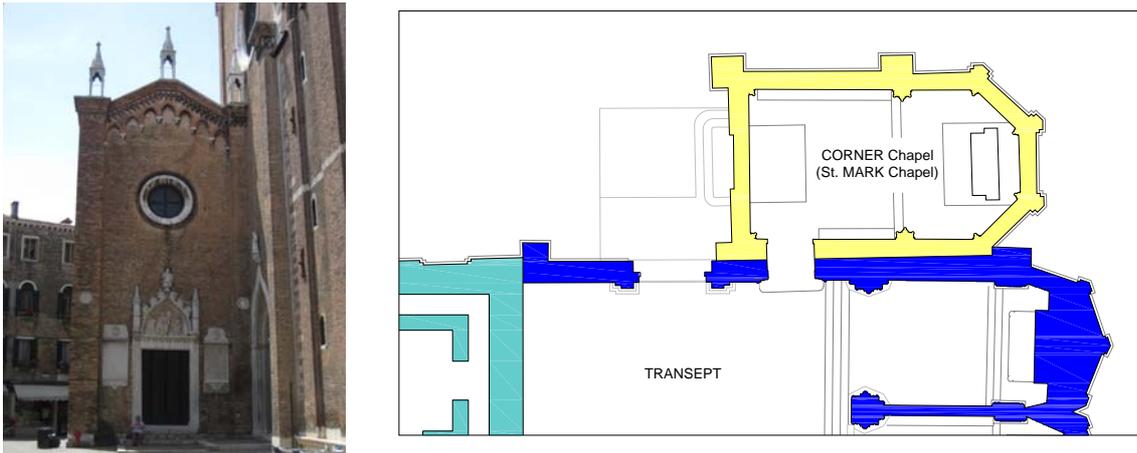


Figure 2-2: St. Peter's chapel – Main entrance and plan view

In 1432, with funding from the wealthy Giovanni Corner, the chapel of *San Marco* was erected against the chapel apse at the left hand side of the transept (Figure 2-3).

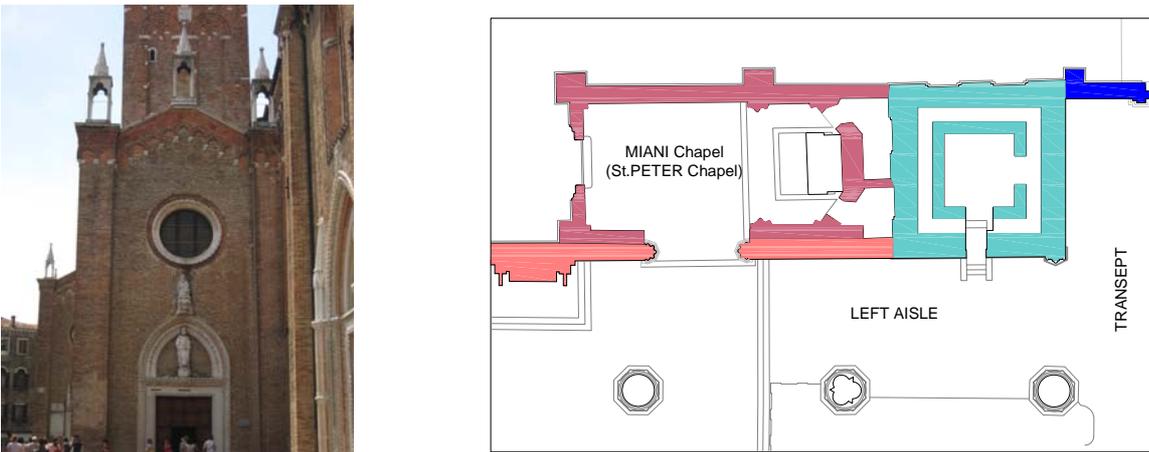


Figure 2-3: St. Mark's chapel – Main entrance and plan view

A third addition to the main body of the church, most likely constructed by the second half of the fifteenth century, is the Pesaro's Chapel (again with a pentagonal apse), on the western side of the Sacristy, which was raised during the same period of the Basilica (Figure 2-4).

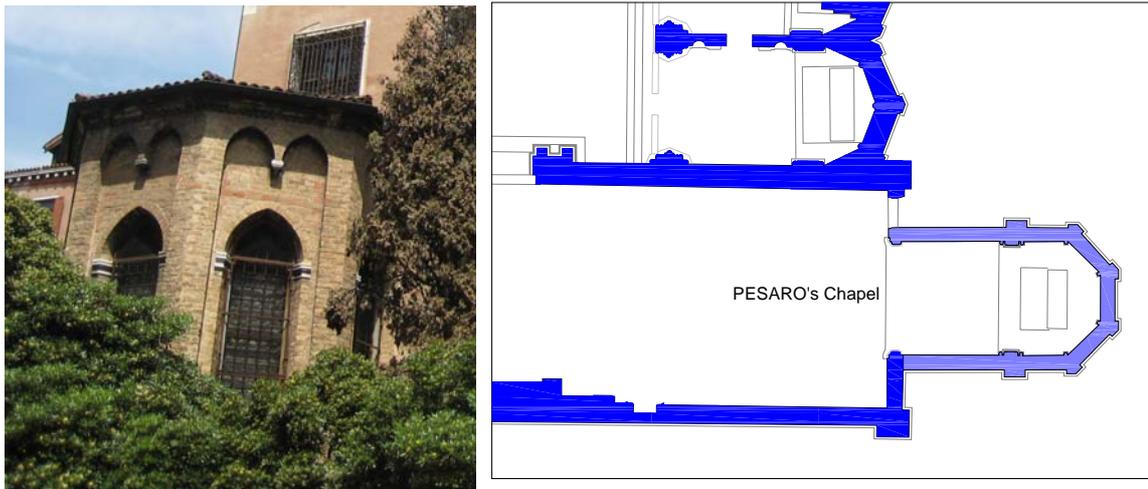


Figure 2-4: Pesaro's chapel – Main entrance and plan view

Historians hold opposing views regarding the date of completion of *Santa Maria dei Frari*. Among the many, one such hypothesis considers the church construction had to have ended around 1428 because at that time a bridge providing access to the main entrance was itself being rebuilt. Yet other historians postpone considerably the conclusion of the work, assigning it to the last decades of the fifteenth century. This hypothesis stemming from black and white paintings of Venice dated 1483 which depicts laborers working on the church roof. Despite the various theories it is however certain that the church was consecrated on 27th May 1492, as a plaque commemorating this event resides on a pillar which divides the first and second chapels on the right aisle.

2.1. *The Basilica*

The Basilica is today one of the biggest churches in Venice and holds the status of “Minor Basilica”. It is situated in the “Campo dei Frari” in the heart of the district of San Polo (Figure 2-5). The church is dedicated to the “Assumption of Mary” and is home to numerous pieces of art including two masterpieces by Tiziano.



Figure 2-5: Location of the Basilica of *Santa Maria Gloriosa dei Frari* in Venice

The present church is brick-to-face view Italian Gothic style, measuring 102 meters long, 48 meters in the transept and 28 meters high. The schematic structure of the buildings of the Venetian Mendicant, particularly the *Basilica of the Frari* comes from inland Venetian Mendicant monuments which the prototype is *S. Lorenzo di Vicenza* (1280). The Friars constructive system is encrypted in this building. This particular method was adopted in the first half of the fourteenth century by the *Frari* and *SS. Giovanni e Paolo's* churches. The elements, all of Lombardic origin, are characterized by modulus plans with the transept opened in chapels, by the presence of ribbed cross vaults, cylindrical pillars with octagonal capitals and polygonal chapels.

It is a shaped Latin cross classic example of Gothic church, to a Gothic it might be called "Franciscan", avoiding the luxury of spires, pinnacles and flying arches, instead using materials such as terracotta with emphasis on harmony, beauty and simplicity of its lines (Figure 2-6). Three aisles with pointed arches are based on six powerful columns per side and are connected to each other by tie rods covered with wooden crates (which were changed over the centuries).

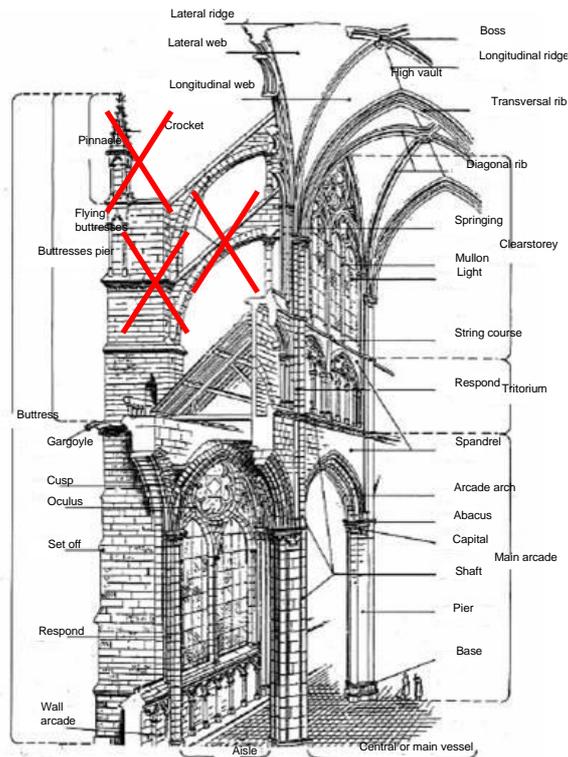


Figure 2-6: Typical luxurious elements in a Gothic church, not present in the Frari's Basilica

The *Basilica of Frari* is notably famous for the masterpieces it houses. An extraordinary wealth of monuments and tomb alter pieces, made by great artists, particularly from the Renaissance and Baroque eras reside here. Together with the church of Saints John and Paul (SS. Giovanni e Paolo), from the fifteenth century, it has assumed the role of Pantheon of the *Serenissima*, accepting burials of distinguished representatives of the Republic and representatives of powerful Venetian families together with many other local famous individuals among them the great neoclassical sculptor Antonio Canova. Along the walls and chapels a great number of sepulchral monuments and works of art can be admired; among the paintings, very known are the two altar pieces of Tiziano: the Assumption (1516-1518), which immediately captures the visitors' attention, and Our Lady of Ca' Pesaro (1518-1526).

2.2. The Bell Tower

Beside the impressive Basilica the bell tower stands (Figure 2-7). At 64 meters high it is the highest in the city second only to the one in *San Marco* square. It was initiated by Master Jacopo Celega in 1361 and ultimately completed by his son in 1396, as the plaque reads at the base of the tower.

The walls are brick made (terracotta) while its base is set in blocks of carved Istria stone. The foundations supporting the tower are independent from those of the church as it was required that they be built deeper into the ground to withstand the larger load but are made simultaneously with those of the contiguous Basilica's walls, presenting similar characteristics, namely techniques and materials.

The bell tower is a twin-tube structure similar to that of *San Marco*, with the inner walls lightened by eight arch shape openings along with a rectangular one at the top. The external walls, following the Gothic fashion, have small openings along each side of its height. Instead of classical stairs between inner and outer walls there are brick ramps to allow climbing the belfry.



Figure 2-7: Bell Tower

The outside walls of the bell tower are divided into three sectors by marble frames which distinguish the three different areas. A further inspection at the top of the left aisle easily make it possible to identify the distinction between areas in the northern side of the first level: a further confirmation that the bell tower was born as autonomous body. The top structure of the bell tower is built in an octagonal shape, with walls adorned with arches; the original conical spire was destroyed by lightning in 1489, as evidenced in literature.

2.3. Construction phases

The erection stages of the ecclesiastic complex can be summarized by three main construction phases:

1. 1361 -1396

The first construction phase, between 1361 and 1396 corresponds with the construction of the bell tower. Initially the bell towers designed foundation construction was set as an isolated body, completely independent of the surrounding buildings.

2. 1340 -1432

The second construction phase took place between 1340 and 1432 and corresponds to the reconstruction of the church in its current form with the partial joining of the bell tower where the transept is. The creation of the church proceeded from West to East, namely by apses from the transept to the main façade at which insisted, until 1415, the structures of the primitive Franciscan church, with opposite orientation. The vertical walls of the church and bell tower, born completely released from each other to allow differentials movements between the two buildings, were locally joined together: both brick cross vaults of the left aisle and of the transept set directly on the walls of the tower (Figure 2-8a).



Figure 2-8: Cross vault set on the tower wall a) and structural joint between chapel and bell tower b)

Building the vaults of the church required a modification of construction philosophy of the bell tower at the North West angle: a partial pillar was introduced in order to support the first stones of the arch that separates the transept from the left aisle.

3. 1420-1450

The third construction phase was set during the early mid-fifteenth century. In 1432 by the will of the bishop of Vicenza *Pietro Emiliani*, the chapel of St. Peter was built, between the wall of the bell tower and the church in the south-eastern corner. The new construction incorporated part of the east wall of the bell tower and was structurally released from the Basilica and the tower (Figure 2-8b).

Further investigations of the northern wall of the chapel have revealed its constitution which is 75 cm in thickness and more importantly that throughout its length it simply leans against the wall of the church without any determining constructive links between the structures in their elevations. The same kind of lacking connection is recognizable at the discontinuity joint between the bell tower and chapel along the external south wall.

2.4. Structural interventions

The *Frari's Basilica* and its bell tower, for the long span of years in which the construction works took place and for the lack of sufficient static, needed restoration interventions. While not many supporting documents exist which would certify the work of restoration period between the sixteenth to the eighteenth centuries, documents do exist for the nineteenth and twentieth century efforts (Figure 2-9).

Three dimensional drawings reporting the structural phases are reported in Appendix A.

Note that there have been a number of less important changes which have not been discussed as their significance to the major restorations were minimal (such as restoration of the paintings and artefacts linked to the Basilica). For further detail, the reader may refer to the publications quoted in the references.

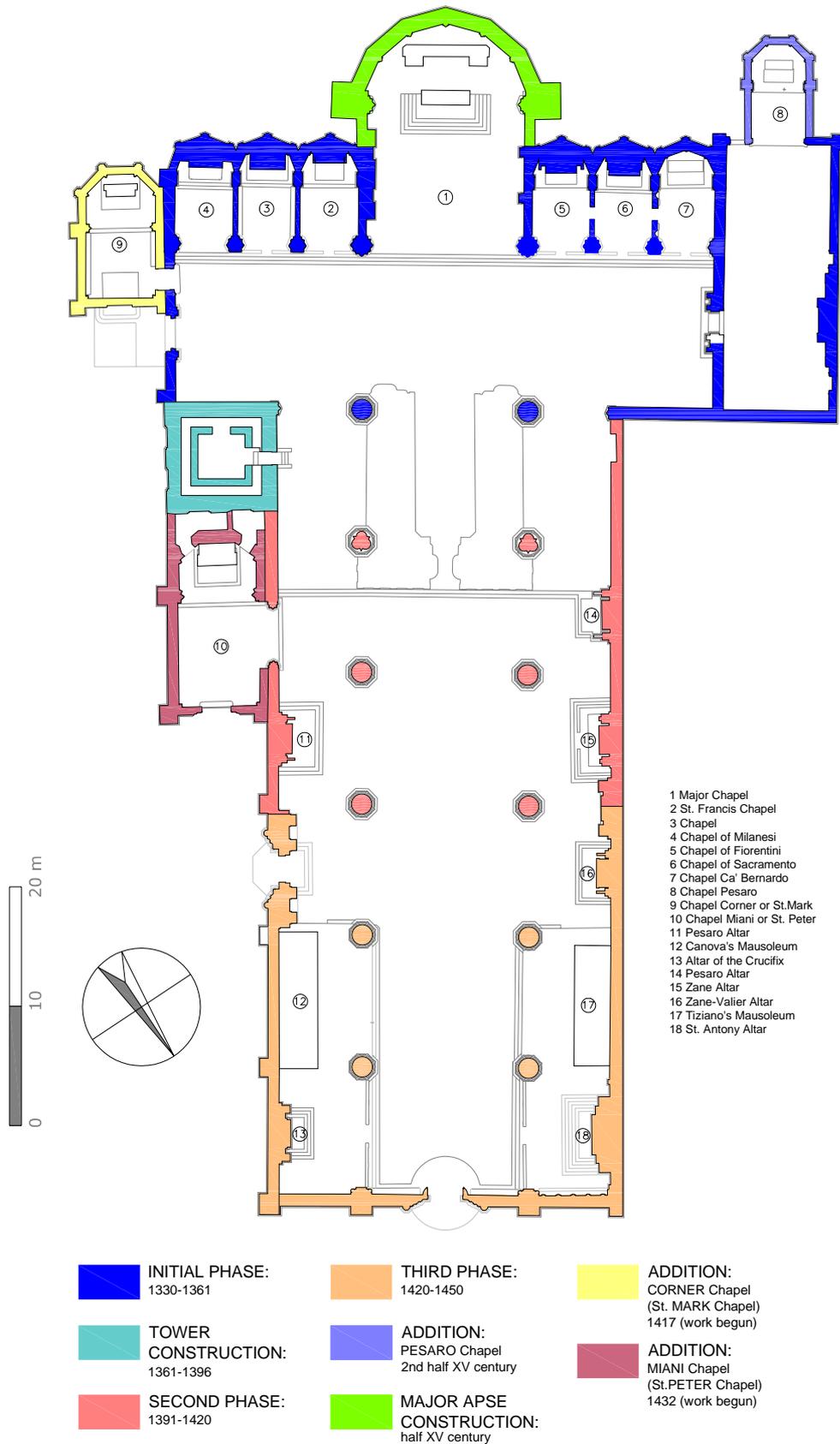


Figure 2-9: Construction phases of the Basilica, bell tower and chapels

3. DAMAGE SURVEY

Any intervention of structural consolidation on a monument of great value and significant architectural complexity as the Frari's Basilica must be preceded by a phase of investigations to define in comprehensive and detailed manner, structural and mechanical properties of masonry load-bearing walls. These diagnostic investigations led by the *Landscape and Architectural Heritage Office of Venice*, started from a detailed reconnaissance of the complex to highlight the crack pattern and the major portions of detached or degraded masonry. The next step in the investigation utilized the technique of the flat jack tests for the analysis of the tensional state of particular masonry load-bearing walls. This analysis of the tensional state, in addition verifying the static behavior of the structure, was particularly valuable for the calibration of the mathematical finite element model through a comparison between the measures observed experimentally and the results of the model.

This chapter will highlight diagnostic stages which preceded the intervention on the bell tower and describe the framework of the crack pattern of the ecclesiastical complex.

3.1. Visual inspection

Visual inspection is one of the most important tasks to be carried out for structural diagnosis. The inspection should be carried out with the aim to study not only the causes, but also the future consequences of the problem. This is definitely the first process that had been undertaken in the initial analysis of the problems of the Basilica. In a first phase, the careful visual inspections should allow to:

- describe completely the structure, in terms of geometry, history of construction and structural changes;
- take safety measurements and to indicate restrictions to the use of the structure during the inspection works;
- define the inspection equipment;
- identify competences and responsibilities for the inspection works.

During past centuries technicians were either not aware of the phenomena that led the structures to suffer in such way or were simply not capable to solve them. Nowadays experts are more inclined to get to the root of the problems and solve them.

A good visual inspection can be of two kinds: direct or indirect.

The first method includes all the features that include in situ measurements of the whole structure with mechanical equipment (such as measuring tapes) and then, in a second stage, transfer the data to reproduce plans, elevation and section of the object. On the other hand the second method embraces all the modern techniques of 3D elaboration data (such as laser scanning, 3D photo-rendering) that reduce the work in situ but make the work heavier during post-processing (Figure 3-1).

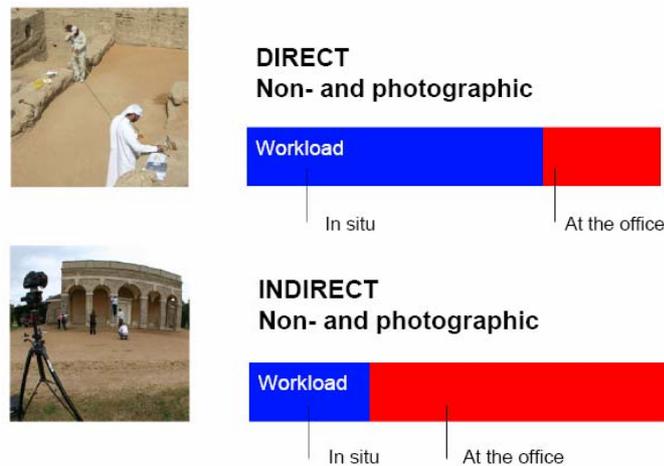


Figure 3-1: Relation between workload in-situ and at the office

The method selection is thus a “time and budget” demand.

For the purpose of the Frari’s Basilica, an imposing structure, it is difficult to implement the use of scaffolding to perform an accurate crack pattern survey. Carrying it out was therefore not simple without the assistance of expert rock-climbers, who looked from close range the walls of the tower and reported on the geometrical drawings all major fractures, noting both the length and width of the opening. The accurate drawings on which the crack pattern was noted were from an accurate three-dimensional survey through the use of software for the photo rendering. It is a quite demanding procedure, but the results are without a doubt of great relevance. It is a powerful tool especially because when a good 3D render is implemented, professionals can have easily access in terms of measuring dimensions, cracks or openings directly from the virtual computer image. In fact, after the finalization of the 3D picture a high quality image can be obtained. This picture can be rotated or scaled in order to have the object “ready to hand”. Furthermore, the image can be easily exported to CAD programs and therefore be modified (Figure 3-2)



Figure 3-2: Examples of 3D rendering images

The photogrammetric survey has some advantages compared to the traditional one, such as a more accurate resolution of the object and the chance to have perfect measurements exactly at the time investigated.

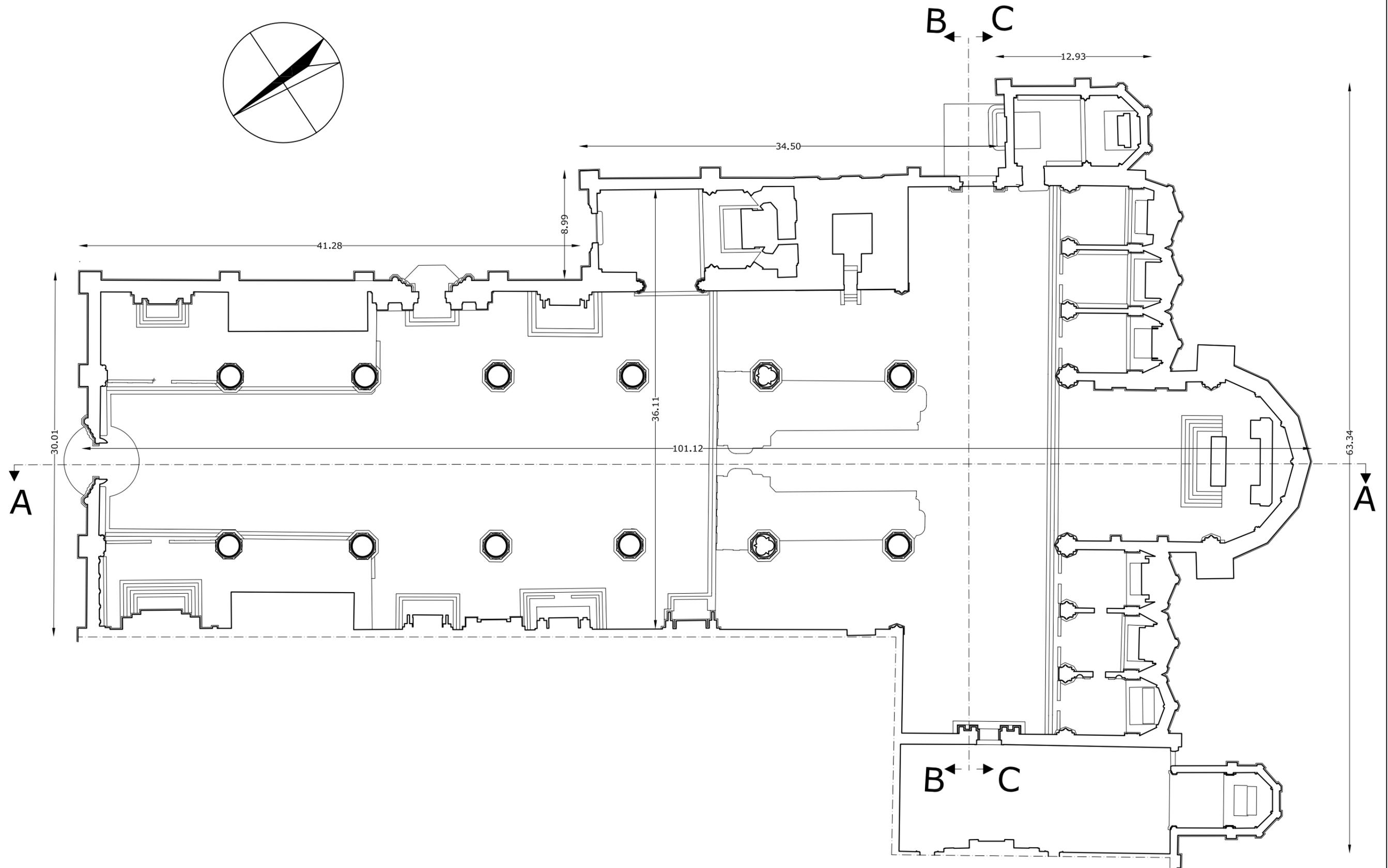
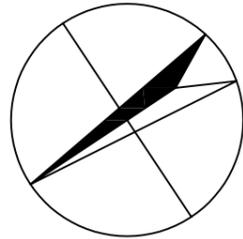
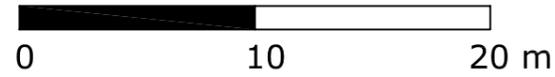
The survey of the bell tower was carried out in 1990 and included the four elevations of the tower, two cross sections and twelve plan views, approximately one for every five meters of height.

This survey is also important because it included the tilting situation of the tower which occurred up to 1990; this data was important so as to better calibrate the finite element model and to clearly understand the different soil pressure due to the uneven distribution of the weight.

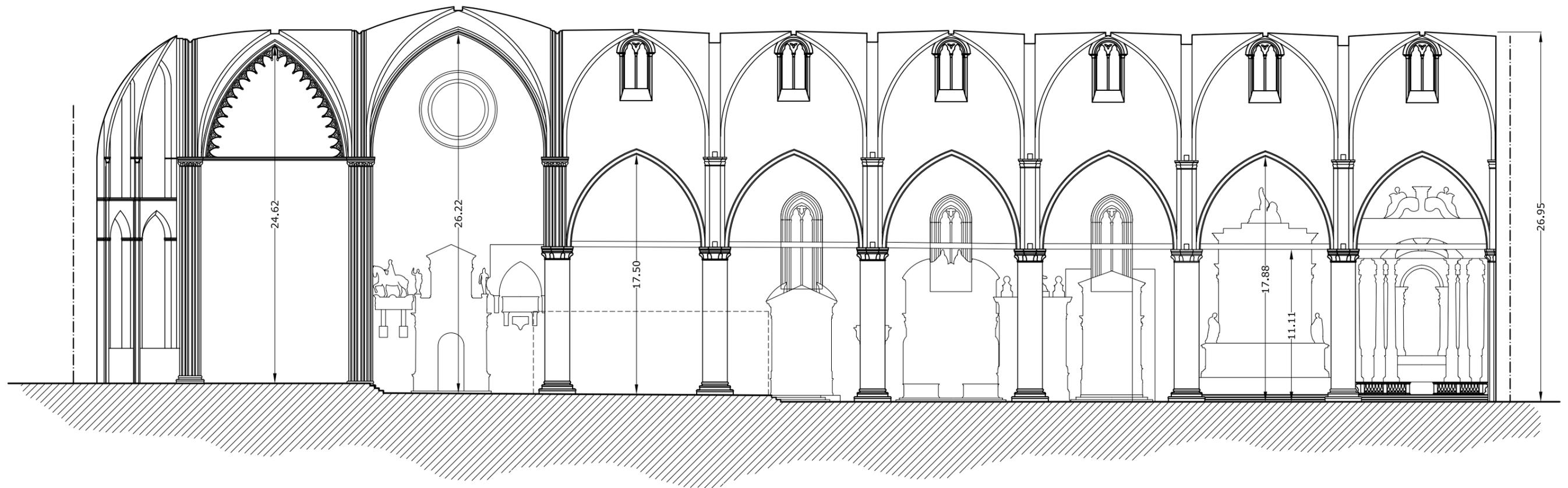
At the same time as the geometrical/photogrammetric survey of the tower, the Basilica had undergone the same treatment. A plan view of the church became available; the longitudinal section (towards west), and two cross sections at the middle of the transept (one facing south, the other in a northern direction). However, all these sections achieved from the geometrical survey could only be drawn up to the intrados of the vaults and walls, not providing any information on thickness of vaults and walls, nor on any structures above the present vaults.

Drawings from the survey of the Basilica are displayed in the following pages, composed by plan view and three cross sections. Moreover, they have been of some help attempting the implementation of the whole finite element model and to complete the 3D Autocad reproduction of the Basilica, which is represented in the next pages.

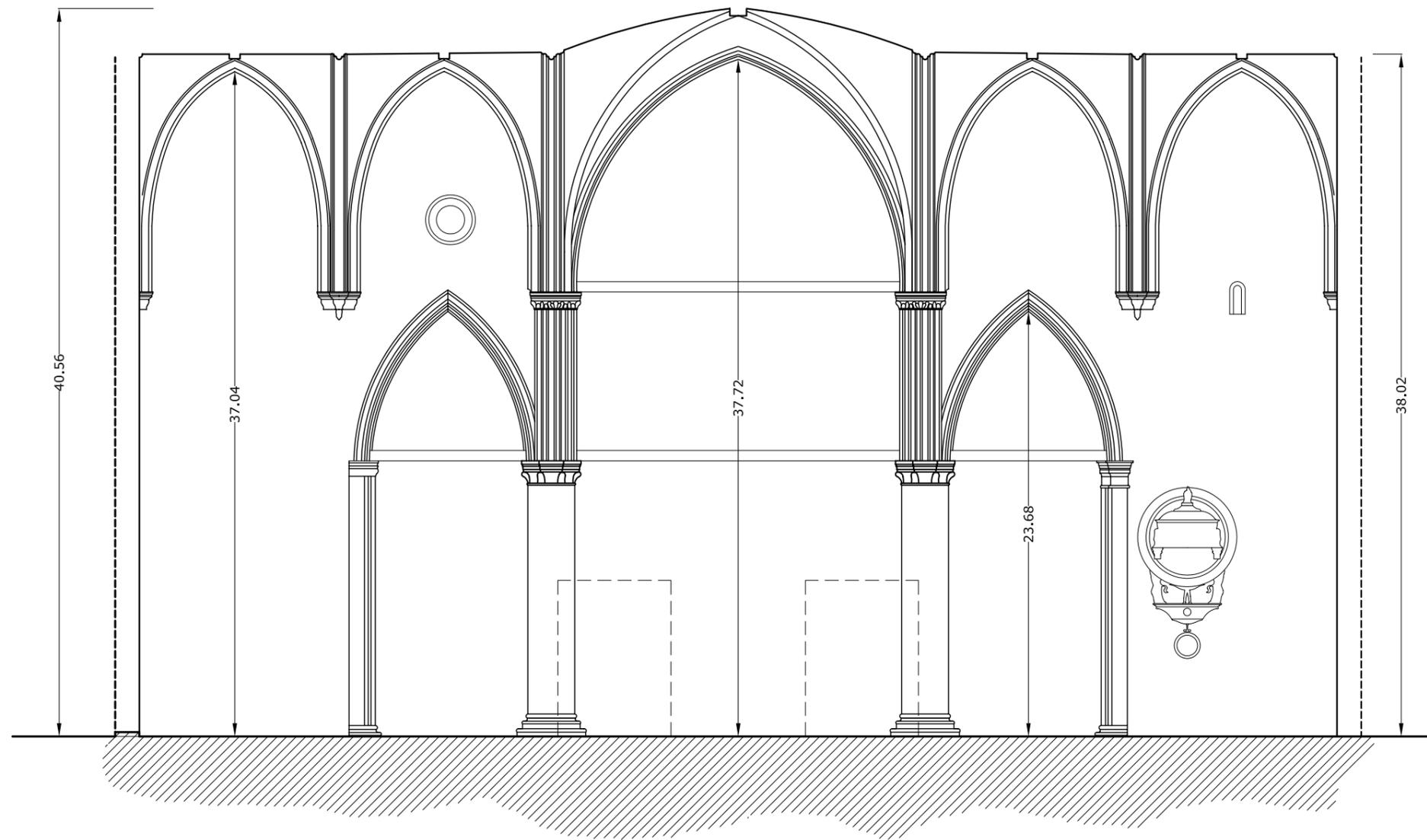
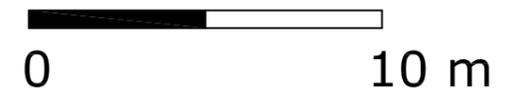
Plan view of the Basilica



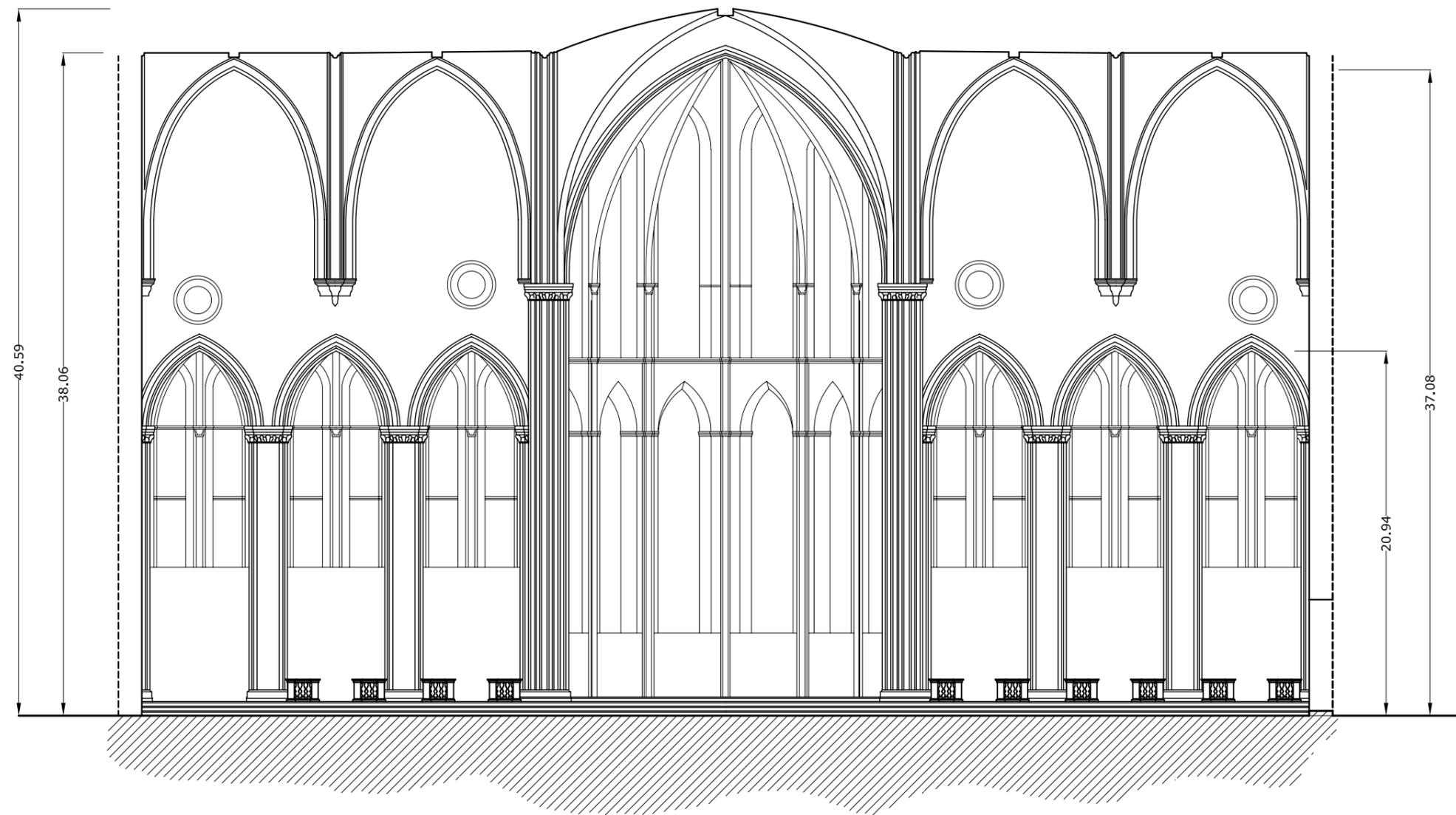
Section A-A

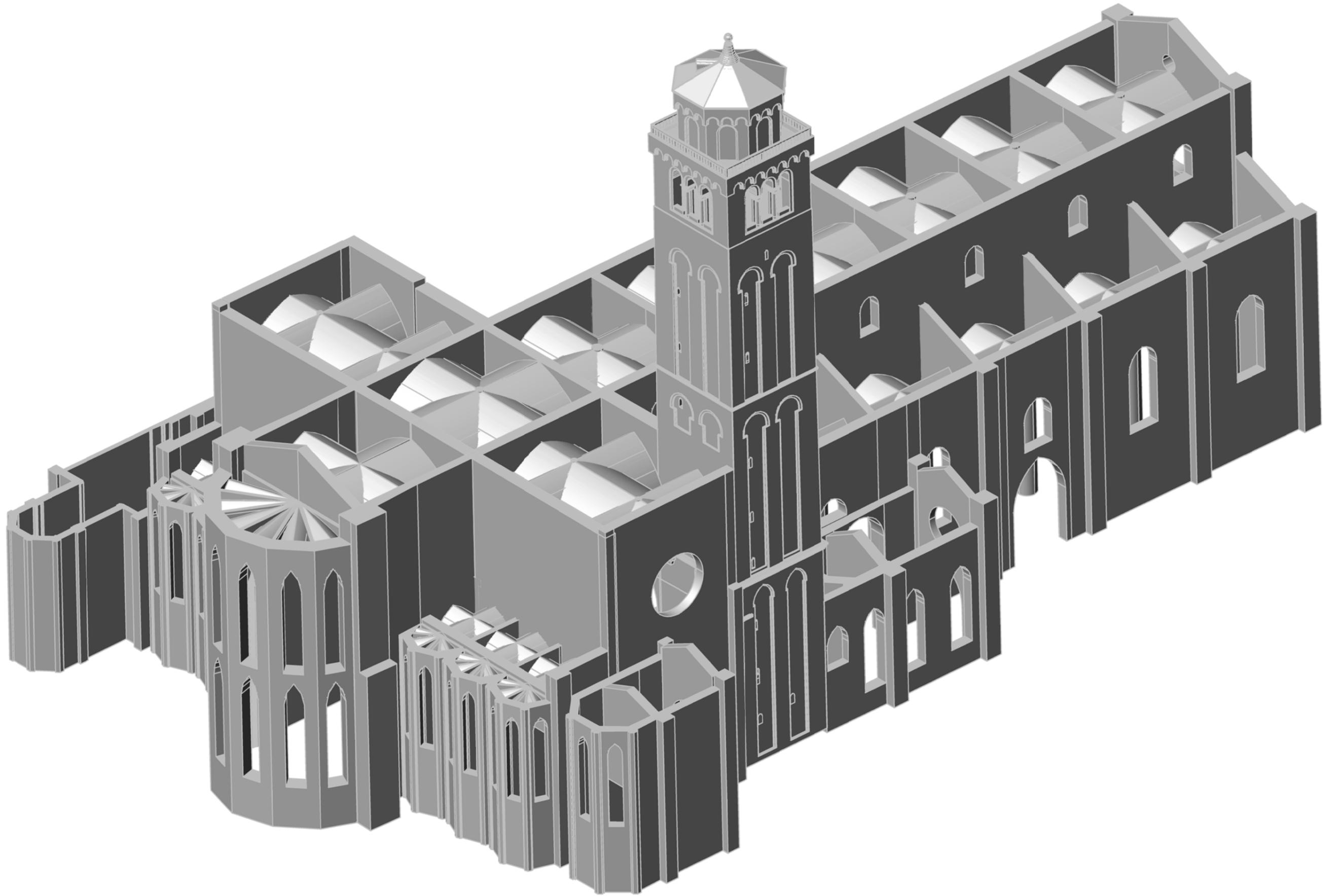


Section B-B



Section C-C





After 2000 as research and studies progressed by the *Landscape and Architectural Heritage Office of Venice* on the behavior of the bell tower, new surveys were done and put on paper. At this point, the geometrical survey carried out was of major value, in particular the assumptions on the foundations of the bell tower and façade of the left hand side of the transept which were affected by interventions in the early years of twentieth and the second half of nineteenth century. On this topic, further evaluation will be performed in chapter 5.

3.2. Monitoring

Another important step to execute a viable damage survey is the installation of a monitoring system, preferably automatic, to detect in real time the behavior of the building.

For historical buildings, monitoring helps identifying if the damages (cracks) are changing with applied force and environmental influences. Moreover, it is important to measure the vertical and horizontal movements of structure. Since movement is influenced by temperature fluctuations and moisture changes, both are to be measured internally and externally. In some cases, measurements can be used to check temperature sensitivity of the devices. Following, the description of the monitoring system is reported.

Crack monitoring:

Crack-meters are used for crack measurements and are installed across joints or cracks by installing a pair of anchor stems. Pre-positioned holes are then drilled on each side of the crack or joint. The expected resolution was up to 0.005 mm (5 μ m) and measured up to 25 mm (\pm 12.5mm). During the first phase of the work, in 1995, the outdated glass indicator instruments were replaced (they were used at the beginning of the XX century), and some of the damages were controlled by installing removable mechanical strain gauges basis (Figure 3-3a).



Figure 3-3: Removable a) and fixed b) crack meters

Other devices were implemented in 2003 before beginning the design phase of consolidation works.

Crack-meters, located on 7 different locations, determined that the cracks were active (Figure 3-3b).

Locations of the crack meters: for deciding on the number of sensors & meters (as well as for a series of electrical gauges) and their location, the following understandings of the structure are highlighted:

1. the deep severe cracks on the transept wall of the bell tower denote that the tower was still detaching from the church and settling towards the South-East direction;
2. the ashlar of the arch that divide the left aisle from the transept were suffering the splitting of the tower and with a tendency to detach;
3. tower wall and perimeter wall of the Basilica (orthogonal to each other), cause of the previous statement, tended to separate;
4. the relieving arch, which constitutes a preferential thrust line, was of difficult interpretation and it may have contributed to the widening of nearby cracks (Figure 3-4).
5. the need to understand the load that affected the provisional tie rod in order to acquire knowledge on the entity of the loads the bell tower transmitted to the Basilica.

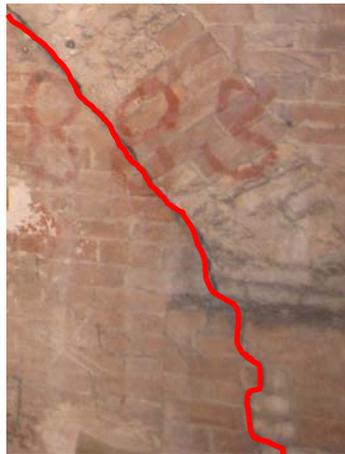


Figure 3-4: Final part of the relieving arch and crack pattern

Hence, with reference to the above mentioned remarks, the following locations to fix the crack meters were decided:

- Two crack meters along the major fractures present in the stone arch that connected the tower to a pillar of the Basilica;
- Four crack meters between the tower and the perimeter wall of Basilica, close to the lateral portal;
- One crack meter on a large fracture among the wall above the arch that connected the tower to the adjacent pillar;
- Four electrical gauges on the provisional tie rod.

These were tied to two stainless steel pins on both sides of the crack were checked and secured to the wall by epoxy resin. The connection of the devices with metal pins were executed through

spherical junctions that allowed the movement of the instrument in three orthogonal axes thus avoiding arising of flexural components in the transducers (Figure 3-5).

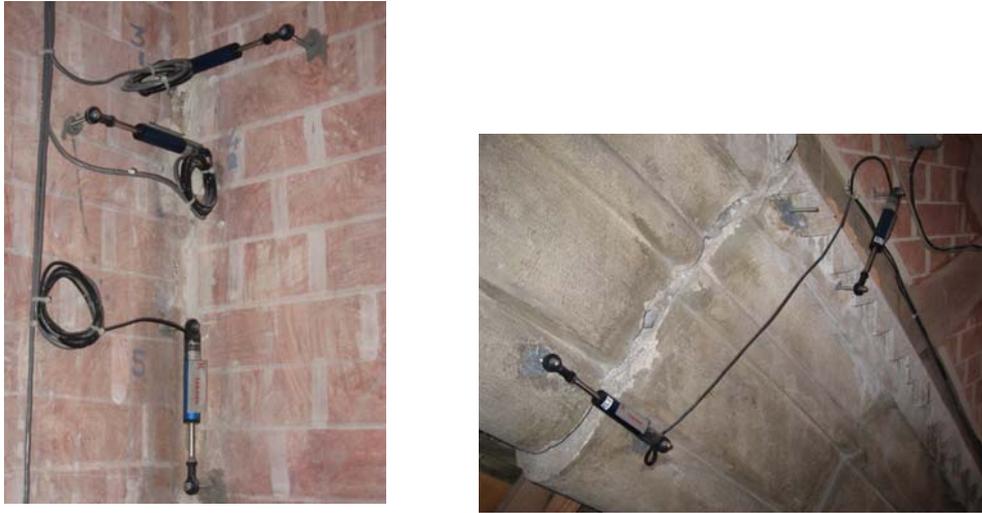


Figure 3-5: Crack meters

Tilt monitoring:

With respect to tilt meters, they are used to measure both horizontal and rotational movements. They can be cable free and be directly fixed to the bell tower. An analog/digital converter and digital radio could be integrated into the tilt meters to achieve real time data.

Location of the tilt meter: Since the only main tilt is found on the bell tower, it was proposed to place one tilt meter within the tower itself. Thus it was decided to complete the structural monitoring system with new tools capable of monitoring real-time deformation processes of the Basilica's bell tower during the execution of labour and at the end of the same, to evaluate their effectiveness.

In particular, a direct pendulum (PD1), able to detect the two horizontal components of the displacement of the top of the tower was installed in the inner tube of the bell tower. (Figure 3-6)



Figure 3-6: Direct Pendulum

In early stages of the structural monitoring of the bell tower, in 1902, benchmarks were carved on the corners of the stone pillars and on the masonry walls to create the basis for accurate survey of the differential settlements. After the already stated foundation consolidation in 1902, this system allowed for analysis the settlements between the tower and structure of the Basilica and to verify that the vertical displacement of the internal wall of the tower (inside the Basilica) was higher than outside. It was then clear that the work of enlarging the foundation stone (only on the outside part of the foundation towards *Campo dei Frari*), had only partially reduced the sinking of the tower reversing the direction of the out-of-plumb and bringing the bell tower to almost lean against the Basilica. Results showed an intense state of progressive cracking.

Later in 2001 it was decided to significantly increase the monitoring system in the most significant points of the church and the bell tower through the installation of other fixed benchmarks (Figure 3-7). These devices were very useful for the execution of precision levelling measurements repeated on a regular monthly base (see chapter 4 for more details).



Figure 3-7: Modern (left) and carved (right) benchmarks

The number of total stations is eight (lettered A through H) while the cornerstones on walls are nine (1-7, 15-16): see in this regard Figure 3-8.

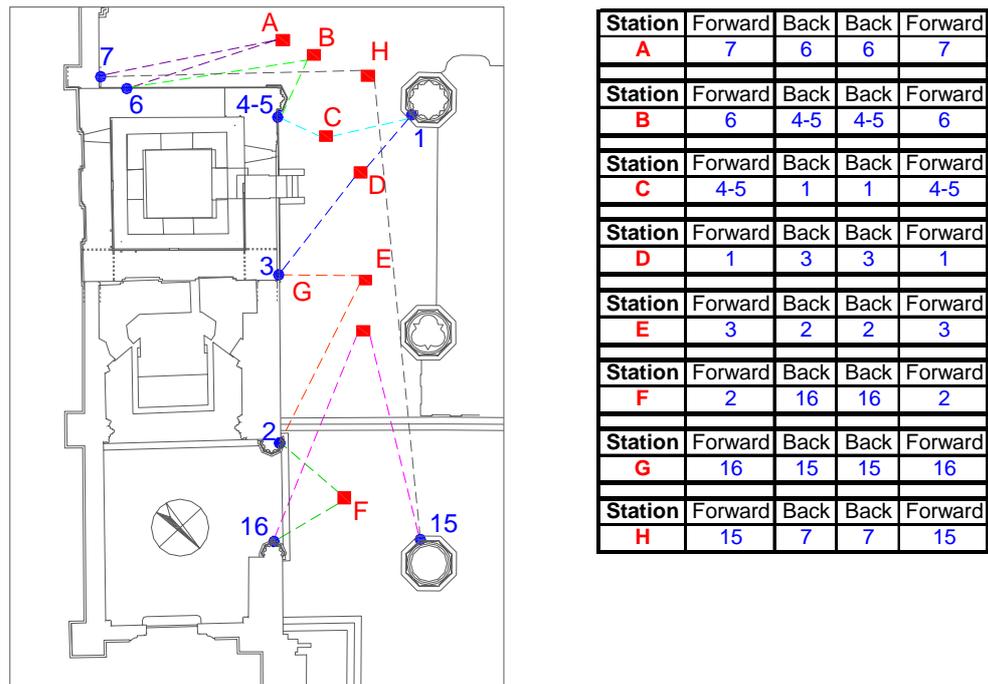


Figure 3-8: Precision Topographical survey (plan view)

To measure traction variation on the metal tie rods, three electrical strain gauges were applied on them, which resolution is 0.1 $\mu\epsilon$ (Figure 3-9).



Figure 3-9: Electrical strain gauges

Temperature monitoring:

The monitoring system was completed by two temperature sensors that detected the air temperature. The temperature sensors were located in such a way that the internal building temperature and the external ambient temperature were recorded.

Location of temperature sensor: The locations of these sensors were directly related to the location of the tilt meter. Thus the decided location was along the tilt meter, placed near the end of the PVC tube that protects the pendulum wire. The accuracy expected was ± 0.1 $^{\circ}\text{C}$ with a maximum

temperature of 100°C. During the previous stage of the monitoring system, two additional temperature sensors were located inside the Basilica.

Data-logger and Receiver:

The core of the system was the acquisition and transmission of data, the data-logger and receiver. It included a multichannel unit for power supply and acquisition of data from the connected devices, security systems against electrical discharge and modem to transmit data to a remote control unit (Figure 3-10). The radio logger needed to operate as the hub of a static collection system. It had to collect data from radio sensors directly, or via repeaters, storing them in non-volatile memory. The system had to allow utmost flexibility in methods of powering the unit, as well as a variety of options on retrieving data.

Location of the data logger: The criteria on which the location of the data logger was decided were as follows:

- location of an existing power socket;
- close proximity to a maximum number of proposed monitoring systems.

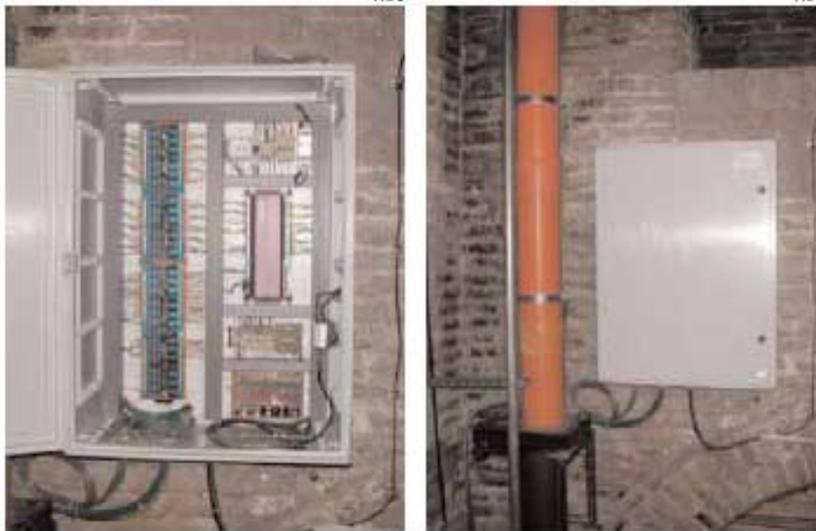


Figure 3-10: Data-logger and Receiver

Hence it was decided to locate the devices of the monitoring system as depicted in Figure 3-11.

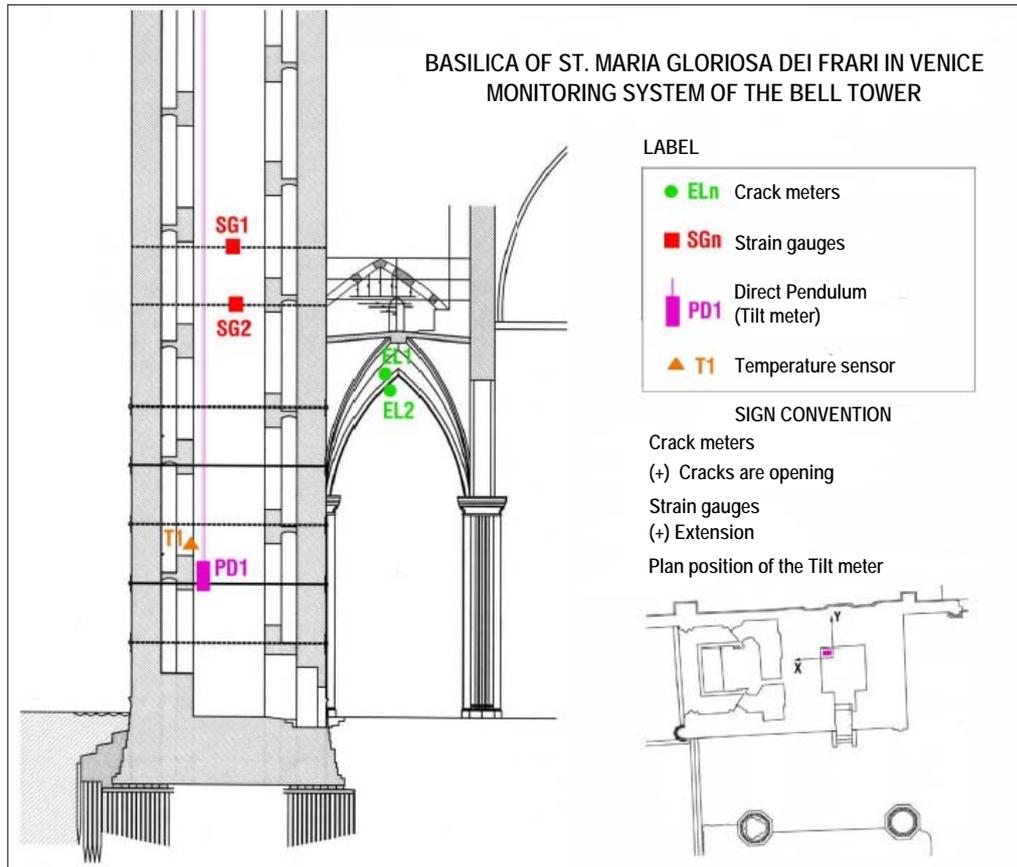


Figure 3-11: Monitoring system (2003)

Figure 3-11 presents the first part of the monitoring system. During the first phase of the works, to gain a better understanding of occurring phenomena, the same system was upgraded with new devices in order to obtain new information about the deformations of the structure.

This also included the geotechnical devices that were added before the execution of the consolidation work of the soil (Figure 3-12).

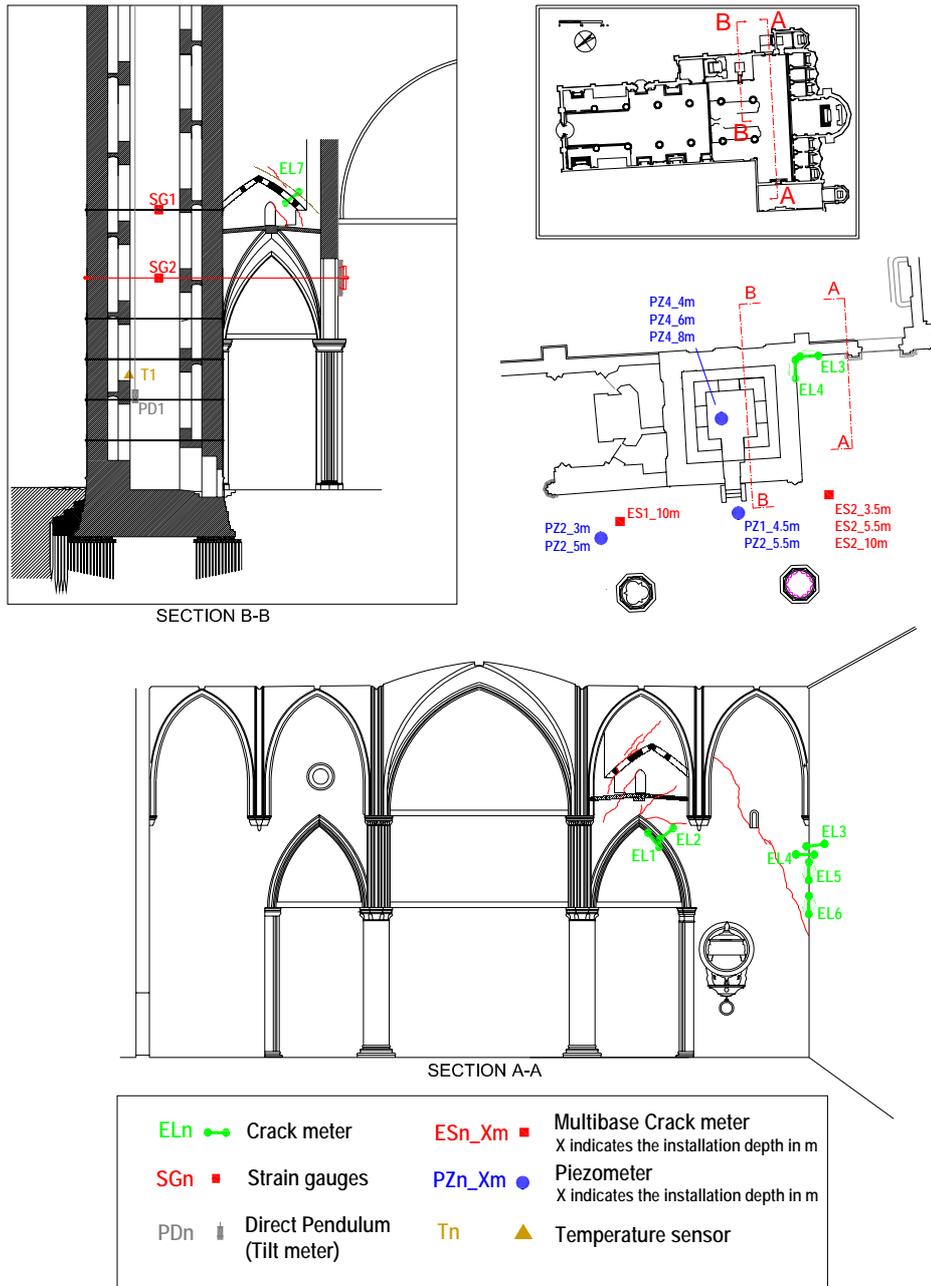


Figure 3-12: Monitoring system (with geotechnical instruments)

All these processes, together with the geotechnical monitoring and the precision topographical survey, made it possible to perform the consolidation intervention of the soil in a gradual and controlled operating procedure.

The main instrument, to which reference was made to control the response of the structures during work, was the direct pendulum. It consisted of an invar steel wire which was set on a shelf at the top of the tower. It was then connected to a counterweight weighing 30 kg immersed in a viscous liquid to reduce fluctuations. By setting a system in the Cartesian plan (X and Y), a pair of photocells identified the position of the wire along the X and Y axis. Along its entire length, the wire

of the pendulum was protected by a PVC tube to avoid accidental collisions and therefore unexpected and unrelated fluctuations caused by air currents or unintended hits.

3.3. Geotechnical investigation

The geotechnical investigation is a topic that would need a further and more accurate chapter within the theme of the Frari's Basilica as it seems to be the key-issue of the problems related to the whole structure. However this point is not the topic of the present dissertation and hereafter it will be briefly discussed and commented in the results of the survey.

The information on the soil beneath the foundations was initially scarce and inaccurate, because they came mainly from documents concerning the work of excavation carried out in the course of consolidation in 1904. This work had in fact brought to light the rock foundation but nothing was known of the underlying wooden piles. It is known however that while considering important monumental buildings of significant mass or with a particular commitment from the static point of view, Venetian technicians used to drop a lot of piles, almost together, to tamp down the soft soil present in the first layers.

Very little knowledge about the local characteristics of the soil beneath the bell tower was known: aside from some available generic data from a couple of cognitive drillings conducted in 1991 within the bell tower. This is why the first and most important step towards the construction of an appropriate geotechnical model of the subsoil which would help designing the most appropriate intervention of consolidation was therefore, to design and implement a detailed geotechnical investigation campaign.

Therefore, investigations are essentially consisted of:

- vertical continuous stratigraphic cores, driven up to a maximum depth of 25.5 m from the floor plan, with undisturbed sampling;
- inclined drilling (5-10° from the vertical), pushed up to maximum depth of 10 m;
- inclined drilling (10-30° from the vertical) to explore the rock mass foundation pushed up to maximum depth of 3.5 m;
- static penetrometric tests (SPT) by piezocone and dilatometric tests for the mechanical characterization of the soil, pushed up to the maximum depth of 20 m;
- dynamic penetrometric tests in sample holes.

Following the trend represented in the lithostratigraphic section (Figure 3-13), drawn along the direction NW-SE and along which were placed the main vertical test, local subsoil can be describe as follows:

- Unit A, Grade level to -3.2 m: filling anthropic material, consisting in sands in silty matrix, little density, locally mud without any stiffness, including lithoids of various kinds (pieces of brick, trachyte and pebbles);
- Unit B, from -3 to -6.8 m circa: dark grey silty clay, with minor stiffness, from normal consolidate to slightly over consolidate, with occasional presence of organic-peat material

and numerous centimetre size shells. Towards the bottom of the layer, silty clay tends to a yellowish colour, indication of oxidation cause of drying phenomenon;

- Unit C, from -6.8 to about -14.5 m: medium-fine clean grey sand, dense to very dense;
- Unit D, from about -14.5 approximately to -16.5 m: alternating silty grey clay highly stratified from soft to moderate consistency, (normal consolidation to slightly over consolidated) and lenses of grey medium fine weakly silty sand;
- Unit E, from about -16.5 to about -21 m: grey sand of average size, locally fine, dense, with rare layers of sandy silt;
- Unit F, up to -25.5 m: alternating of silty clay and medium fine dense sand..

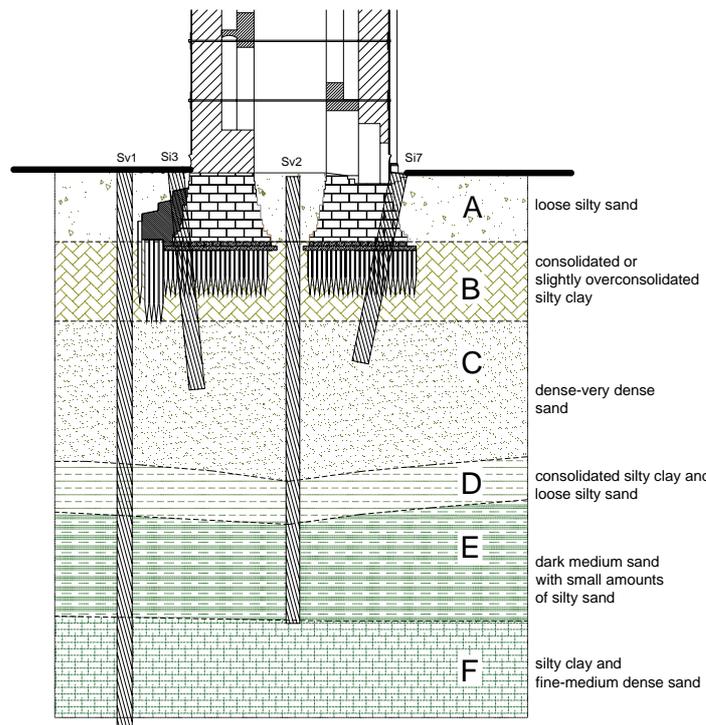
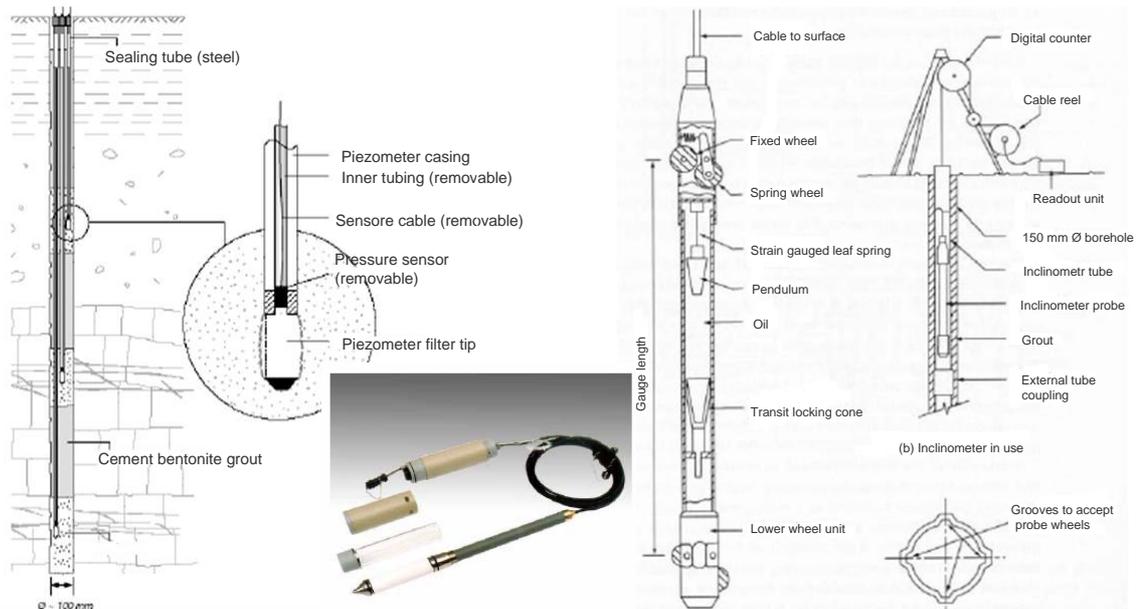


Figure 3-13: Subsoil layers

The level of undisturbed water table has been found to be substantially related to local tidal phenomenon, characterized by daily fluctuations with an average of ± 0.5 m compared to the sea level, identified in about one meter below the local grade level.

Piezometric cells:

The verticals piezometric are composed of three electric piezometers installed by pressure means (Figure 3-14a) with penetrometric equipment and driven to different depths into the soil.



**Figure 3-14: Typical multiple piezometric pore water pressure measurements in different depths (a)
Borehole inclinometer (b)**

By monitoring the data collected from these devices it was discovered that the increases compared to the value of hydrostatic pressure due to water table, recorded concurrently with perforations and injections, had meaningful values only in the piezometric cells placed in clay (because of its low permeability). Piezometric cells placed inside the sandy soil, in line with its granular composition and increased permeability value for this typology of soil, have instead highlighted neutral pressure values in substantial agreement with the undisturbed groundwater level, without suffering any consequence from the work carried out above.

Multi-base deep strain gauges

The multi-base deep strain gauges, installed in sample boreholes, have three measuring devices in three different depths of the soil (3.5/5.5/10 m from the Grade level), for the measurement of the vertical displacement between the measurement basis and the head of the instrument and, consequently, between the basis of measurement.

Borehole inclinometers

Boreholes inclinometers assist in determining magnitude and direction of displacement in potential or active landslide or (in this case) in monitoring safety upon completion of injections. Measurements were done almost monthly or whenever significant work within the influence radius of the devices was carried out (Figure 3-14b).

3.4. Diagnosis

The first important step to carry out after the conclusion of the damage survey is the crack interpretation. It is essential to identify whether the cracks are structural or non structural and to establish whether they are active or inactive in order identify the behavior associated to the crack type. An active crack is related to an unstable phenomenon that can increase important structural

movements and it might be related to a temporary overload case or, as often occurred for historical constructions, for a foundation settlement, which redistributes the stresses in the structure into a new equilibrium condition, for which most likely the structure is unable to withstand. As a result of an accurate crack interpretation, proper alarms can be triggered, important collapse behaviors identified and serious structural problems in the medium to long term can essentially be warded off.

3.4.1. Structural defects

In the bell tower, most of the cracks inferred from the damage survey presented a vertical trend with limited extension; a concentration of cracks close to the corners of the tower could be seen, possibly due to local concentrations of loading. Closer hair cracks could be seen at the belfry, most likely related to the later substitution of the old belfry with a new electrical one (Figure 3-15).

Especially on the South-East and South-West elevation, the zones where typical phenomenon of *spalling* are highlighted (which is the detachment of one layer of a material higher than 3 mm thickness).

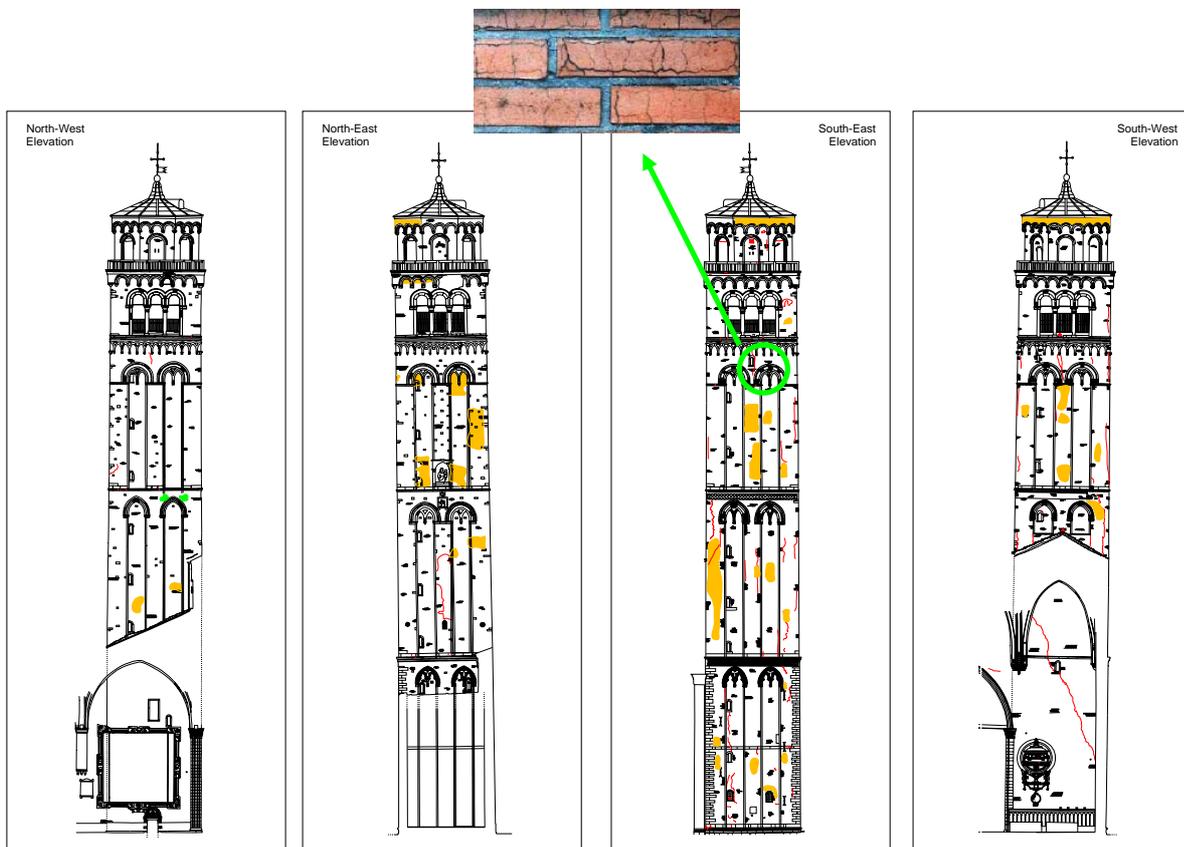


Figure 3-15: Crack pattern on bell tower elevations

Further cracks cannot be seen from outside, as in recent years major works of local rebuilding were performed. Nevertheless these interventions left a visible mark on the masonry and a virtually clear interpretation on whether the masonry had been replaced or not. In Figure 3-16 a clear example of the damage which occurred to the interface layer of the two structures is shown: the rock stringcourse on top of the Basilica (green spot) was buried in the wall of the tower. After the

different settlements occurred over the past centuries and after a small redistribution of stresses among itself (rock is a weak material, not good working in tension) it literally exploded causing potential hazards to people walking by.



Figure 3-16: Rock stringcourse before and after intervention

The more concerning situation is that depicted inside the Basilica. Most likely cause of the interventions of the beginning of XX century, the changing in behavior of the structure produced the crack pattern depicted in Figure 3-17.

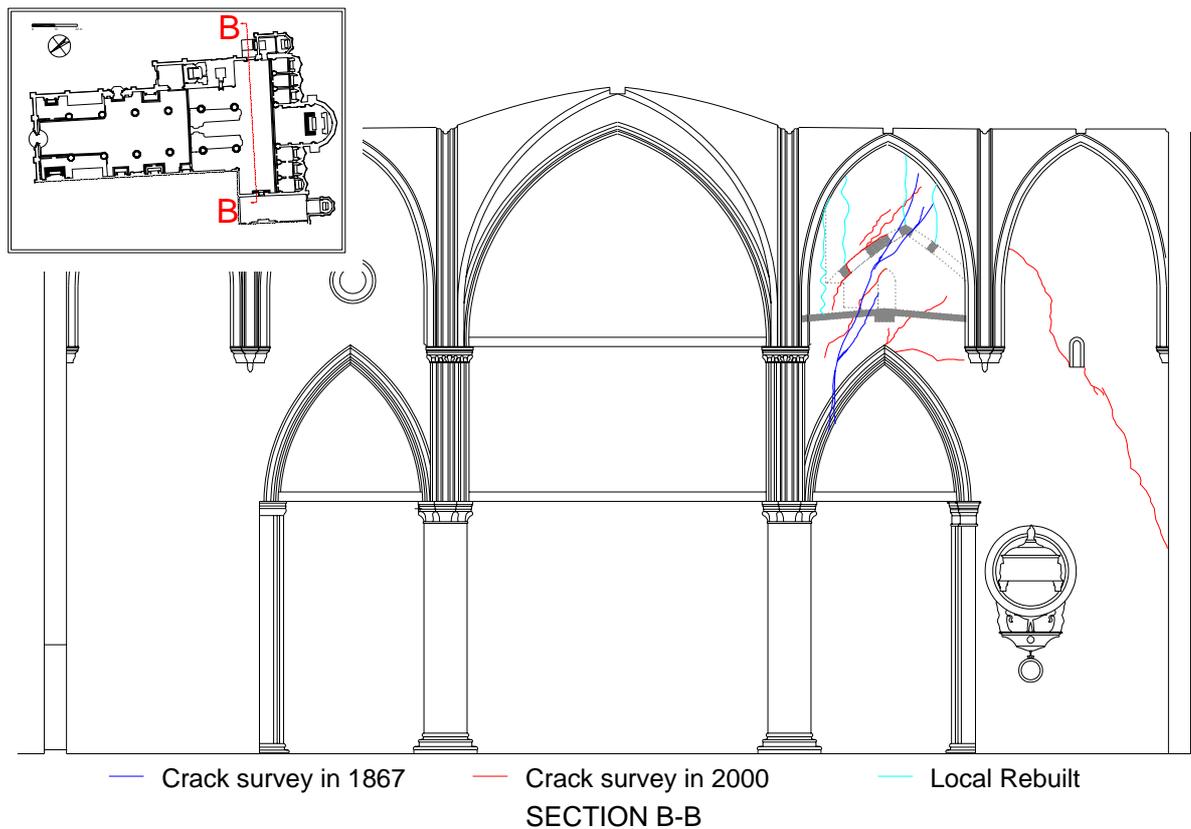


Figure 3-17: Section of the transept

While during second part of the nineteenth century the crack pattern was due to the differential settlement and the out of plumb towards "outside" of the bell tower, after the intervention of the beginning of the past century the new crack pattern suffered from reversal behavior; while the widen of the foundation towards "Campo dei Frari" seemed to have stop the tilting of the tower, during the following decades, and thanks to the geometrical survey, a reversal of trend had been noticed.

The red 45° crack that arises on the bell tower wall towards the transept (Figure 3-17) is clear evidence of the actual trend.

It can also be stated that, performing a simple limit analysis on the building, this fracture could lead to a formation of a macro block defined as "*part constructively recognizable and accomplished of manufactured goods, which may coincide with an identifiable part (also in the architectural and functional aspect, e.g. façade, apse, chapels); it is usually formed by several walls and horizontal elements connected to each other to form a block which is considered as an unitary constructively part, although generally independent and not linked by complex Construction*"¹.

The typical collapse mechanisms are determined on the base of the observation of the collapse modalities of the existing buildings, collected in abacuses. These abacuses are divided depending on different construction typologies and the determined mechanisms are schematized with kinematic models, based on equilibrium conditions, which provide a collapse coefficient for the elementary mechanism (i.e. the seismic mass multiplier that leads the element to failure).

A range of index had been elaborated (Lagomarsino et al., 1997) for assessment of damage produced by the earthquakes (e.g. Umbria and the Marches), based on sixteen indicators, each representative of a possible kinematic mechanism of collapse for the different macro elements. The combined assessment of the level of damage and of the construction characteristics allows quantification, through an index, of the damage produced by the earthquake and definition of a vulnerability index of the church. Nonetheless it can be deduced that considering the significant mass of the bell tower and its location, (Venice, which falls within the "zone 4" of the new Italian code, thus a zone with a PGA of 0,05 g²) there is no need to carry out a seismic analysis for the above mentioned macro element as the result of its safety is rather positive than negative.

In the same Figure 3-17 another interesting point can be noticed: the crack pattern that arose during the past centuries has been hidden by meaning of local rebuilt (cyan lines) and this intervention is not always clearly readable with a visual inspection.

Climbing the belfry gives access to the extrados of the cross vaults; from here it is possible to inspect the relieving arch (Figure 3-18) that had been constructed in the past centuries and for which purpose and reasons are still not clear. Apparently this relieving arch had not been built for the entire thickness of the wall and to better investigate its behavior, a certain amount of flat jack tests have been conducted among its length.

¹Lagomarsino S. "A new methodology for the post-earthquake investigation of ancient churches"

² Despite being part of zone 4 (PGA=0,05 g), Venice has a PGA of 0,07 considering a more accurate microzonation

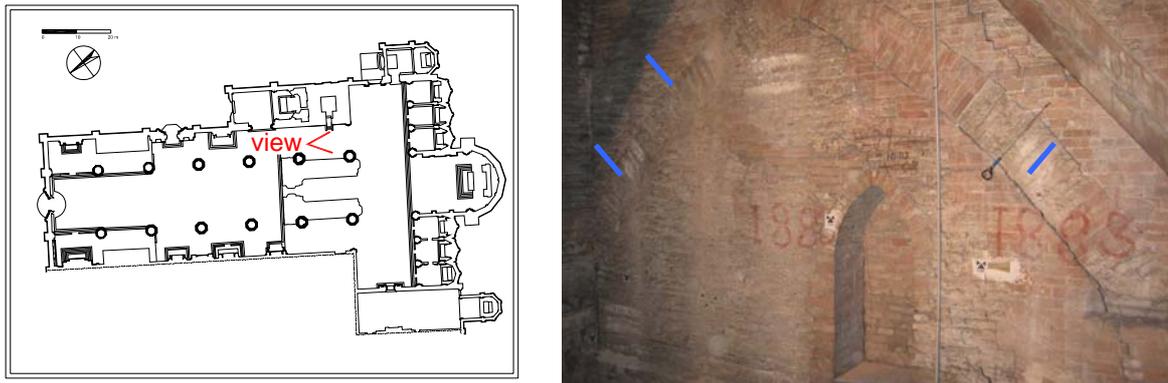


Figure 3-18: Relieving arch and position of the flat jack slots

From the same room it can be noticed as well the attempt to scarf together the wall of the transept of the Basilica and the South-West wall of the tower; as it can be seen with a closer view, the scarfing bricks are not connected at all with the orthogonal wall. This ancient effort of connecting the two structures is a clear indication that they were built independently (Figure 3-19).



Figure 3-19: Scarf among the structures

While the first phase of the work was focused on fixing the differential settlements of the buildings, the aim of the second phase is to try to set up the pristine configuration between the two buildings, which is their physical and therefore mechanical structural separation.

However the procedures to perform this separation may be complicated and of difficult decision on where to physically perform the cut and the most difficult will be the interpretation of future behavior. More information on this topic is given in chapter 5.

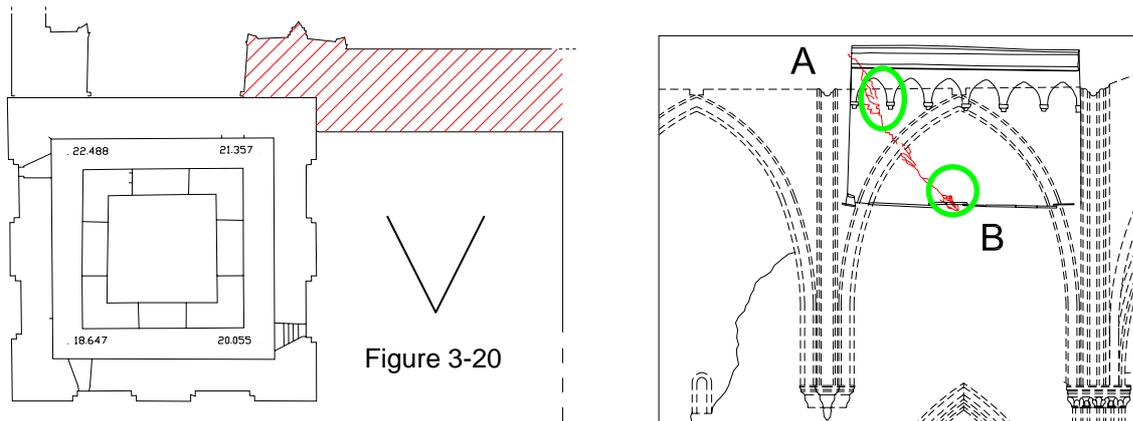


Figure 3-20: Outside transept crack A) and B)

A further hazardous aspect inferred from the damage survey is the one depicted in Figure 3-20. It is the external crack that matches the red cracks depicted in Figure 3-17.

These cracks, which follow the trend of the tensile isostatics in the part of the wall that connects the bell tower to the Basilica, explain the formation of inclined struts (between the main cracks) capable of transmitting relevant components of vertical and horizontal forces from tower to church and vice versa. Even if the amount of forces enrolled in the exchange between the structures is difficult to define (cause of high non linear components) their pattern is undoubtedly clear, as depicted in Figure 3-21.

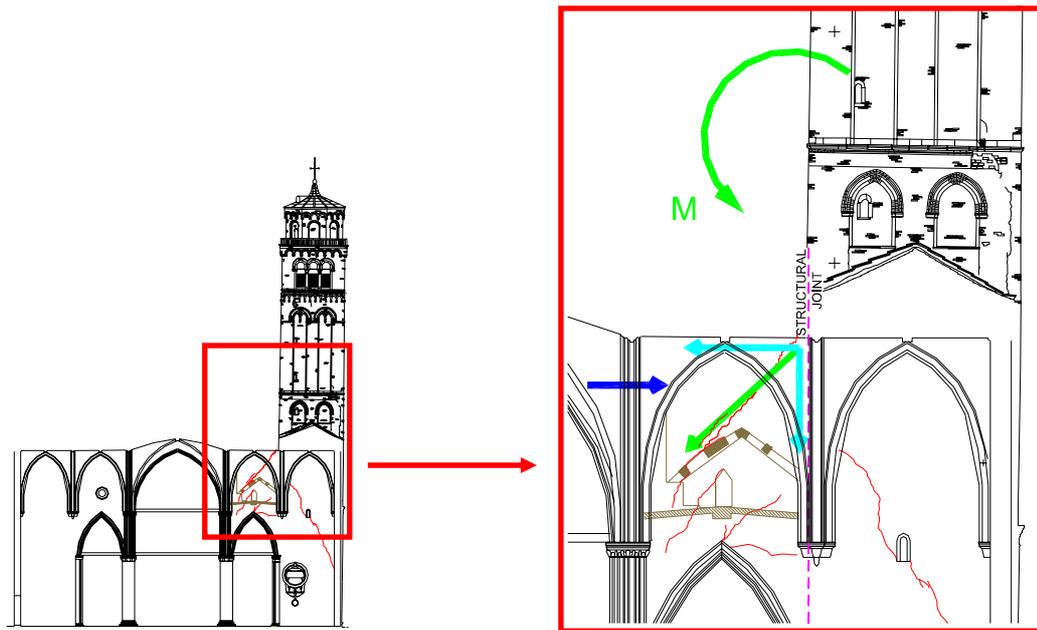


Figure 3-21: Loads exchange pattern

An increase of the compression state of the column and therefore an arising flexural moment on top of it, are the consequence of both eccentric vertical load (increased) and horizontal load. Thus the pillar is highly subject to a strong deformation which is clearly demonstrable with the values of the flat jack test performed on different locations of the pillar. In fact the column is highly compressed at its base towards inside the Basilica (white –Figure 3-22) and a relatively high tensile state in the opposite side of the base cross section (purple). The formation of the cracks induced a redistribution of the stresses among the structure, thus an increase of vertical force in the column induced a reduction of vertical force in the tower. But such exchange of forces had far worse effects in the column than in the tower (due to reasonable matter of cross sectional area).

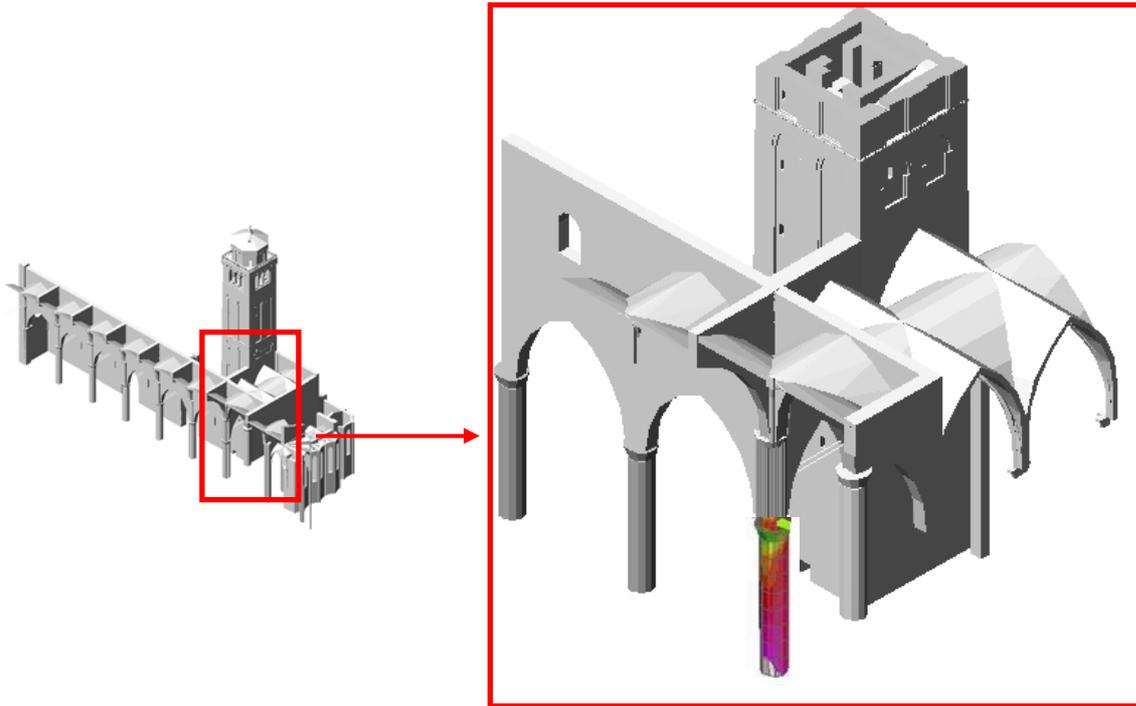


Figure 3-22: Actual behavior of the column

Moreover the stronger stiffness of the left hand side of the transept wall (blue, Figure 3-23), compared to the remaining part on which the horizontal force of the tower acts (cyan, Figure 3-23), let the weaker part of the masonry wall, already fractured, become even more seriously compressed. This phenomenon induces a state of *bulging*, which is namely a deformation of a wall consisting in a deviation from its original form, which the shape of deviation is roughly a curve, as can be noticed in Figure 3-24.

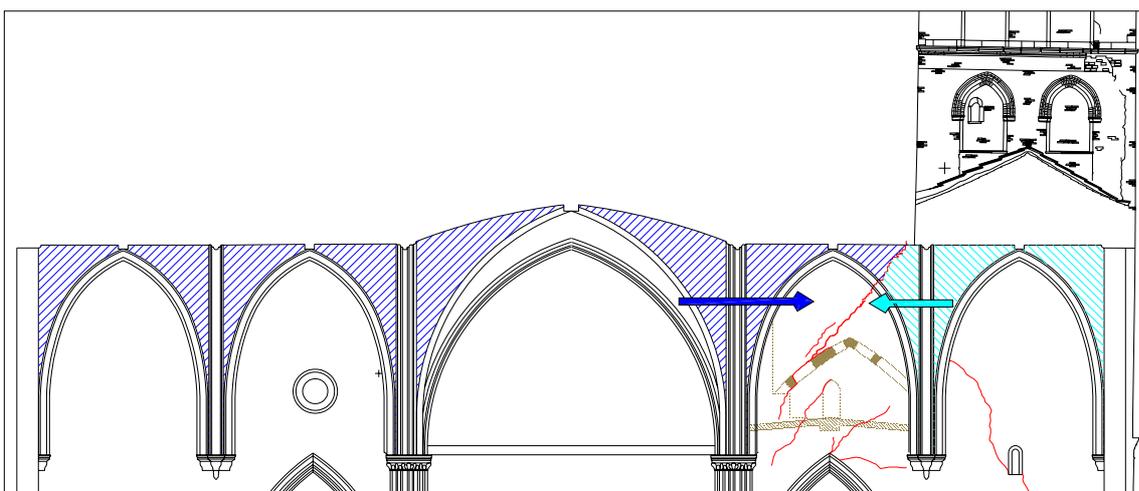


Figure 3-23: Possible explanation of the cracks in the upper part of the transept wall



Figure 3-24: External transept wall crack pattern

In the same position again a primitive joint can be noticed between the two structures and the attempt to scarf them together (Figure 3-25). This surely is a late effort as the local rebuilt (on the other side of the wall depicted in the Figure 3-19) does not allow a wooden rule meter to go through the joint but it stops after about 40 cm in confirmation of the subsequent execution of repair works.



Figure 3-25: Scarf masonry bricks and original structural joint

As the topographical survey confirmed, the major settlements were concentrated in the localized zone of the tower, St. Peter chapel and the intersection between the tower and the transept. However some other cracks, of minor importance, can also be seen in the main façade, below the main rose window and at the base of some pilaster (Figure 3-26).

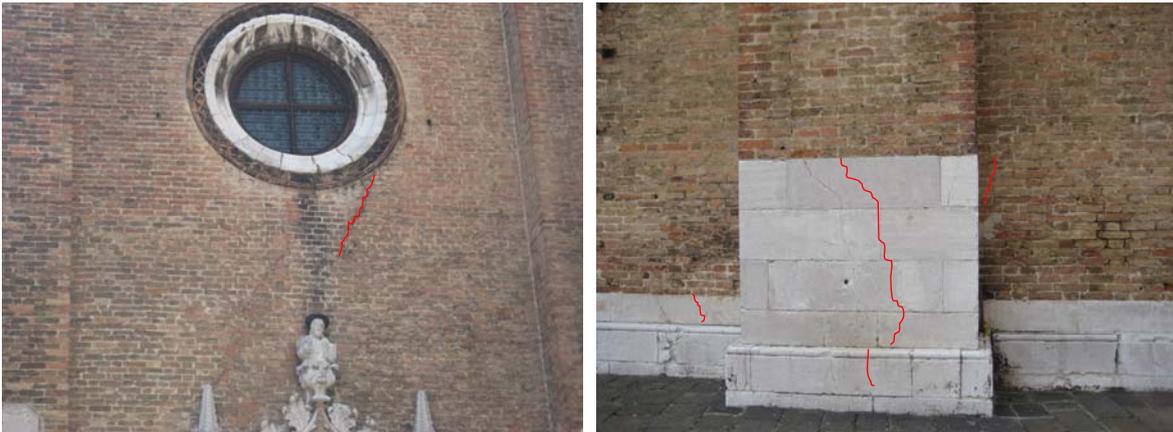


Figure 3-26: Minor cracks on the front façade

These cracks do not seem to be imminently dangerous and could be related to phenomenon of *exfoliation*. This consists of the layering (several layers) of material with an originally not laminated structure, or they can be also related to hair cracks as their width is not significant.

A more interesting phenomenon that could be noteworthy is the so called *creep* that appears thus causing vertical parallel (sometimes thin) cracks which eventually leads to collapse. In this case, this phenomenon is joined by the occurrence of *chipping* which consists in one or more fragments that tend to separate and be broken from the edge of individual bricks or natural stone blocks (Figure 3-27). A closer look at this kind of trend highlights the chance of an increase in the phenomenon of *sanding* that can be seen in the same Figure 3-27 and occurs when the binder gets less cohesion.



Figure 3-27: Creep associated with chipping and sanding

3.4.2. Non structural defects

Not related to structural phenomenon but only to the old age of the building, other pathology can be assessed to the Frari's Basilica. These worries are not to be disregarded as they testify to the importance and long lasting life of the building, nonetheless they induce experts to perform accurate analysis on phenomenon that occur at micro-levels and that arise especially in a salty environment such as the Venetian one.

Mortar seepage marks:

At many points along the masonry joint of the external walls one could see dark seepage marks (Figure 3-28).



Figure 3-28: Mortar seepage marks

This could be the dissolution and leaching of the components of hydrated mortars.

The phenomenon behind *dissolution* and *leaching* of the components of hydrated mortars can be related to excessive hydration as well as dehydration (i.e. through absorption, leakage, and percolation or splashing of water). The dark colour can therefore justify the mixture with different kinds of biological activities such as pigeons' excrement or it can represent the creation of a *Patina* which is a thin surface layer of the chemical transformation of the materials.

Vegetation Growth:

Due to the high concentration of moisture in the Venice environment, this phenomenon could have been more relevant in the Frari's Basilica. In fact there is no evidence of vegetation growth in the opening joints of the Basilica. The only vegetation spot encountered during the visual inspection is in the North-West wall of the bell tower (the one facing the Basilica) and in a pretty high location thus its removal is difficult. Nevertheless it can hardly be seen from people walking by (Figure 3-29).

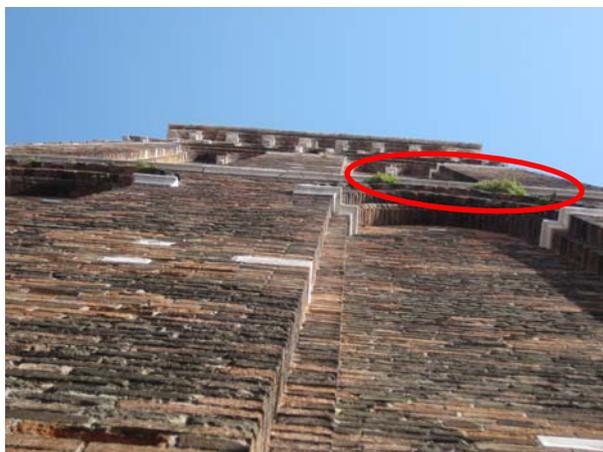


Figure 3-29: Vegetation growth

Pigeons:

The presence of the excrement of pigeons may have become cause of the deterioration of masonry elements, in that it can affect the composition of the clay bricks and trigger chemical reaction within its components (Figure 3-30).



Figure 3-30: Possible chemical reaction

Moist spots:

A common trend for the building in Venice is suffering moisture problems.

Air humidity may be a source of moisture sufficient enough to cause damage or to allow processes like salt crystallization to take place. Apart from visible or clearly deducible sources of moisture, it is necessary to examine the possibility for one of the following processes to occur:

- Groundwater/rising damp
- Rain penetration
- Rainwater leakages
- Rain, air humidity

The classical result of the moisture action is the moist spot which occurs when the surface of the material shows zones of darker colour than the original one.

Rising damp, another dangerous mechanism in Venice, has been wisely avoided by interposing a thick layer of Istrian stone (less permeable) between the soil and the masonry superstructure. However salt crystallization can be noticed in various areas of the external walls of the structure (Figure 3-31).



Figure 3-31: Moist spot and salt efflorescence

Moisture related problems affect steel elements as well. A certain amount of steel pieces can be observed all around the structure and all of them, as a result of the aggressive environment, present a dangerous state of oxidation (rusty brown colour). As known, oxidised metal can increase their volume up to 7 times the original and induce unexpected stress in the masonry around (Figure 3-32).

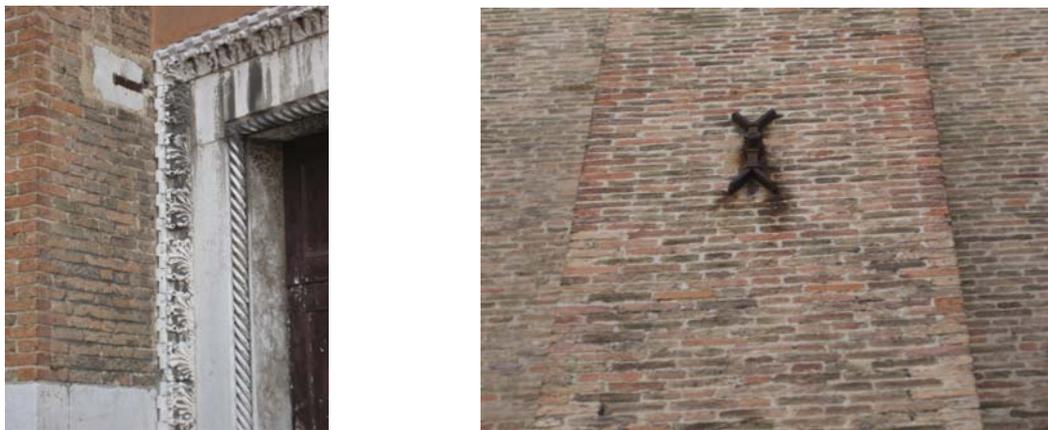


Figure 3-32: Oxidised steel elements

4. NON DESTRUCTIVE AND MINOR DESTRUCTIVE TEST

4.1. Testing in brief

Until 1995 there was no evidence of tests carried out to obtain information about the structure. Documents of past interventions such as the one in 1864 and 1902 (relieving arch between St. Peter Chapel and the bell tower, and the enlargement of the tower foundation respectively), do not reveal specific NDT or MDT beside the geometric survey that ascertained the troublesome out-of-plumb.

Therefore, in recent years, the need to carry out accurate investigations on the structure was undoubtedly facilitated by recent technological developments.

Non-destructive (NDT) and minor-destructive testing methods (MDT) are tools of investigation which can be applied without any or with only small interventions on the object to be examined. These techniques can provide hints of irregularities within the historic masonry structure, which is often inhomogeneous.

Starting from the surface of the object, NDT and MDT offer opportunities to define and circumscribe problem areas, to detect structural differences and to amend the reliability of statistical evidence relative to, or in addition to, selective material extractions and investigations.

The selection of the investigation methods depends on the accessibility of the object and its condition, on the problem to be solved, as well as many other factors, which only the experienced investigator can oversee. The combination of several investigation techniques like ultrasonic, impact-echo, sonics, radar and some stress estimation methods, like flat-jack and hole drilling, can give more reliability for the interpretation of results and for the detection of irregularities like voids, cracks, presence of moisture and/or salt. Moreover these techniques may help out the calibration of a finite element model. The interpretation of the results of such tests is not always straightforward and does not easily apply to further engineer calculations. As a matter of fact the incorrect understanding, assumptions and use of tests can lead to misapplications such as extrapolation of erroneous material property data outside their domain of validity.

Furthermore attention must be paid given that the masonry as an isotropic material within the local environment in which the test is performed; such assumption is therefore unrealistic for many engineering materials but a good point from where to start to obtain first important results on the building behavior.

4.2. Test purposes and test evaluation criteria

Each structural test involves imposing one or more loading conditions on test hardware and obtaining some measurements and other indications describing structural response. Therefore each test conducted should have one or more **purpose** or **reason** why the test was performed. Most evident reasons are how and why a structure, or its assembly, responds to applied forces and other conditions (such as environmental changes) and to obtain specific data needed (such as

strength). Test purpose and evaluation criteria need to be established before a test is performed because they are essential in the planning, organizing and conducting of a test.

Evaluation criteria are also needed because their use provides the necessary evidence and assessment of test performance, and enable test personnel to comprehend the underlying causes for structural responses to interpret test results and select complementary test techniques.

The understanding and insight gained from structural tests may include correlation with analytical models of structural behavior, proving or offering evidence that structures can meet, or not, their intended strength or deformation.

Non destructive tests (NDT) also referred to as non destructive evaluation (NDE) or non destructive inspection (NDI), are tests that do not destroy the specimens that have to be tested. While destructive testing usually provides a more reliable assessment of the state of the test object, destruction of the structural element usually makes this kind of tests more costly and often unfeasible. The great advance of non-destructive tests is that they enable **repetition** of tests. NDT is divided into various methods, each based on a particular scientific principle. These methods may be further subdivided into various techniques. Non destructive testing is a branch of the materials sciences that concerns all aspects of the uniformity, quality and serviceability of materials and structures. Not all NDT techniques are physically capable of detecting all discontinuities, each with its own limitations. The capability of a method depends on the inherent limitations of the method, technique, or procedure used.

Furthermore it is necessary to distinguish between the *effectiveness* and *efficiency* of inspection. An effective inspection is one which finds all the required defects with the required probability of success. On the other hand, an efficient inspection is one which is not only effective for defects but also avoids the unnecessary rejection of minor imperfections. It can be therefore be deduced that inspection reliability includes both effectiveness and efficiency. Hence the choice of inspection equipment is a function of several important considerations. The equipment must have sufficient capability yet be simple to calibrate, maintain and operate, it must withstand the field conditions of the inspection, it should allow ease of signal interpretation and recording, it should be portable.

The solution of very difficult problems cannot be reached with a single investigation technique, but with the complementary use of different techniques.

The types of tests available nowadays are mainly based on the detection of the physical properties of the wall. The flat-jack test provides local measurements and is slightly destructive: nevertheless it can directly provide the values of mechanical parameters. In the case of ND tests, a correlation between the measured parameters and the mechanical ones are usually difficult, but they can give an overall qualitative response of the masonry. At present the most diffused ND techniques are represented by the sonic (or ultrasonic), radar and thermography tests.

Up to now most of the ND procedures can only provide qualitative results; therefore the designer is required to interpret the results and use them to assess comparative values between different parts

of the same. Needless to say that NDT/MDT should be preferred to traditional tests on extracted samples when both types of techniques can solve the problem.

The combination of a series of NDT and MDT is therefore a potential source of data to assist in the decision for structural interventions.

4.3. Test campaigns in the Frari's Basilica

According to the importance of the monumental building at hand, Landscape and Architectural Heritage Office of Venice and experts in charge had scheduled a major plan for the main tests to be carried out. The main objective of the tests performed in the Frari's Basilica was clear since from the start: to provide evidence of the interaction between the tower and the main body of the Basilica. Nevertheless, NDTs or MDTs have been performed at the same time, with the goal to identify and control previous intervention (tie rods) and detect new linings (thickness, quality, behavior).

Some of the tests described in this section are strictly related to the monitoring system described in the previous chapter and they give important feedback which is correlated between each other.

The design project of the test plan includes:

- **Non-destructive** testing *in situ* (sonic test)
- **Minor destructive** testing *in situ* (flat jack tests)
- **Destructive tests** on small non-standard specimens made from samples extracted from the existing buildings (masonry cores).

Investigations performed, divided by structural and geotechnical tests are described in the following sections.

4.3.1. Elastic wave method – Sonic test

The elastic wave method takes advantage of the wave propagation properties of the materials by using two kinds of waves propagate in an elastic body: the pressure (or longitudinal) waves, P-waves, and the shear (or transversal) waves, S-waves.

The method based on P-Wave Speed measures the time it takes for the Pwave generated by a short-duration, point impact, to travel between two transducers positioned at a set distance apart along the surface of a structure. The P-wave speed is calculated by dividing the distance between the two transducers by the travel time.

Ultrasonic waves are preferably used for the study of continuous structures. Masonry structures, which are typically inhomogeneous, necessitate the use of sonic impulses. The equipment required to perform sonic tests consists of an impulse hammer (typically with three tips of varying hardness) to initiate the stress waves and piezoelectric accelerometer to measure the vibrations of the wall, resulting from the propagation of the waves. The surface condition of the tested masonry wall and the desired energy and frequency content of the impulse, dictating which tip will be utilized.

Hammers and accelerometers are connected to an amplifier and an analog-to-digital converter, coupled to a laptop computer. Effects are thus viewed in real time and both the generated impulse waves and the propagating pulse waves are stored. Usually, to obtain better accuracy, the investigated body must be homogeneous and anisotropic.

Sonic testing is a completely nondestructive method and is the most frequently used due to its practicalness of providing qualitative measures to describe the masonry structure.

They consist of transmitting stress waves, within the frequency range of acoustic waves (500 Hz÷10 kHz), in construction materials such as concrete, masonry, wooden elements, etc. The travel time of stress waves through homogeneous and isotropic solids is proportional to the dynamic elastic modulus, Poisson's ratio and density of the medium.

The fundamental relation to comprehend and select the more convenient method is the following formula which correlates velocity (v), frequency (ν) and wavelength (λ):

$$\lambda = \frac{v}{\nu}$$

For a given value of speed, an increase in frequency results in decreased wavelength. This means better resolution in terms of recognition of small defects. Indeed, according to a rule of thumb, to produce a measurable effect, a discontinuity must have the size at least equal to one quarter of the value of the wavelength. As the frequency increases the energy mitigation increases as well causing a limitation on the thickness of the section which can be investigated.

There are various methods of sonic test (Figure 4-1):

- direct (or transparency);
- semi-direct (or radial);
- indirect (or surface).

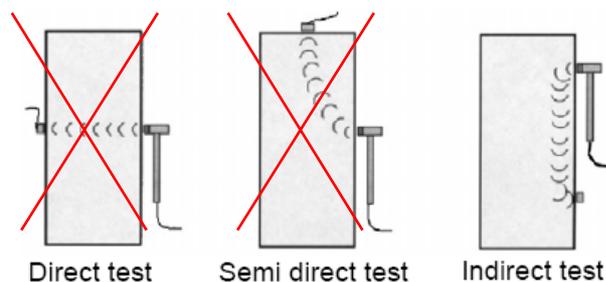


Figure 4-1: Sonic transmission method typology

In the direct test the transmitter and receiver are located on opposite sides of an object (masonry wall) and the sonic wave goes through the entire cross section of the wall following a straight line. It represents the most sensitive type of test, but requires accessibility from two opposite sides and knowledge of the thickness of the wall.

In the semi-direct test, the two points, although on opposite walls, are no longer in symmetrical position, or belong to two unparallel walls. This method introduces a certain amount of uncertainty linked to the length of the transmission line (between transmitter and receiver).

In the indirect test (or impact-echo) the transmitter and receiver are positioned on the same façade. This is the least sensitive test of the three explained thus far.

Regardless of the method, sonic speed, measures the consistency of masonry in accordance with the principle for which the elastic wave tends to spread through area with greater density. For instance, if the wall has a void, the transmission time increases because it increases the effective distance covered by the wave.

Generally speaking, the following Table 4-1 can be used as a reference for the correlation of the mean velocity that can be evaluated in damaged and undamaged masonry specimens and their quality (Berna M., 1993).

| Average Speed | Damage level of the masonry |
|----------------------------------|---|
| V < 1000 m/s | Masonry brickwork seriously damaged or with extensive voids within. Low resistance material |
| 1000 < V < 2000 m/s | Masonry brickwork in poor conditions. Values lower than 1500 m/s indicate the presence of voids and cracks, uneven bricks or joints pattern, not adequate mortar. Values of compression strength can be ranged from 2 to 5 MPa. |
| V > 2000 m/s | Masonry brickwork brick carefully constructed or in good conditions. Values of compression strength can be ranged from 5 to 15 MPa. |

Table 4-1: Correlation between speed and quality of the masonry

In real case studies, it is advisable to apply other non destructive or minor destructive tests to cross-check the qualitative results of sonic tests.

Available sonic test data on the walls of the bell tower are of limited interest, mainly due to the type of tests performed (indirect sonic test and impact-echo). The values obtained from this sonic test campaign can therefore qualitatively characterize the conditions of the walls and be helpful when crosscovering that information with data from other tests.

After a visual inspection it was clear that all the masonry of the bell tower was uniform; the tower had been built in one unique phase, hence the chance to have different layout patterns within massive walls of the tower was low. The investigated walls of the tower were opposite to each other and belonged to the outer tube of the twin-tube structure. The inner tube was of no interest due to its extremely small available area where the test could be performed as each level has four big openings, one on each side (Figure 4-2Figure 4-1).

Indirect sonic test were preferred for two reasons: first because it was a quicker test and mainly because at those heights, scaffolding construction would have been very costly and challenging.

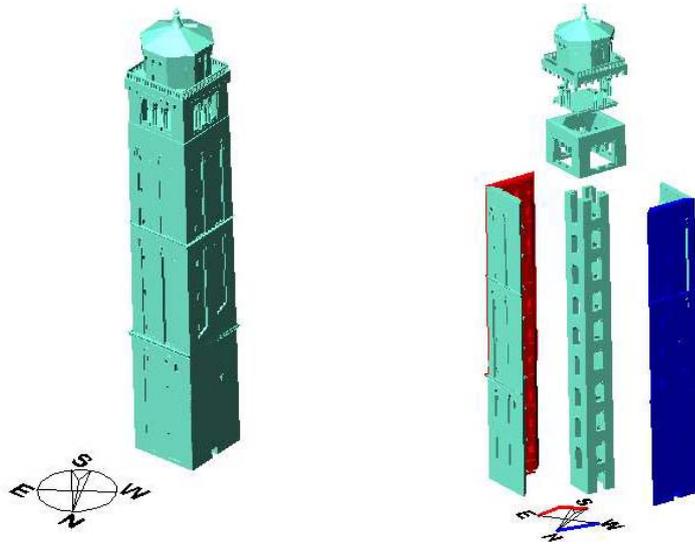


Figure 4-2: SE (red) and NW (blue) walls

Sonic tests were performed in the following locations:

- sonic test 1: wall NW at approximately 35 m
- sonic test 2: wall SE at approximately 25 m

The layout of the test is depicted in Figure 4-3, in which an elevation and a section of the wall are presented. As can be seen, the investigated area of the wall is 3.00 x 2.00 m. Five rows can be clearly noticed in the layout scheme: even rows represent the positions of the transmitter while odd rows represent the receiver points. Transmitters are referenced with letters from A to N while receivers are referenced with numbers from 1 to 6. In the same Figure the path of the wave is also highlighted: holding the receivers in a fixed point (e.g. point 1), by hitting the surface with a special hammer on four different and predetermined points (e.g. points A, B, E, F), the velocity of four different path can be recorded and stored in a portable device (e.g. A1, B1, E1, F1). During the test the operator may check whether the signal is good or not and have a first glance at the stored data. Typically the 50 x 50 cm mesh is usually too large to have a clear understanding of the object, in this case the size fits the specimen to be tested and gives pretty accurate results, as shown in Figure 4-4 and Figure 4-5 where the positions of the tests within the tower are highlighted and the results are shown by means of a tomography image.

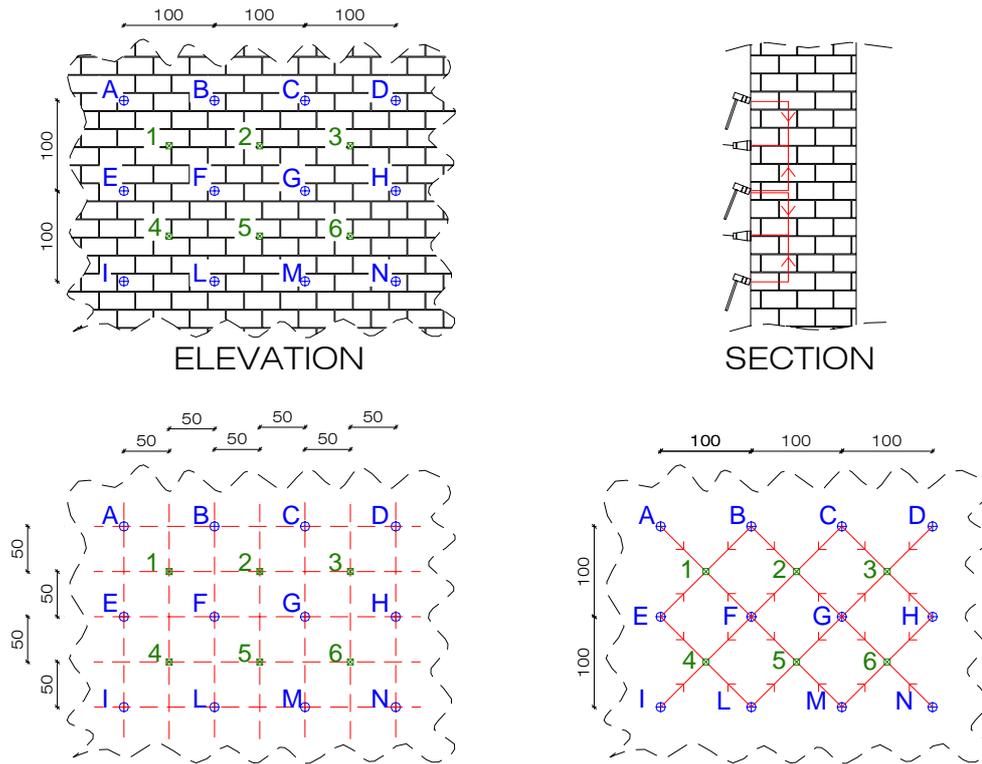


Figure 4-3: Sonic test layout

By referring to Table 4-1 it can be seen that masonry constituting the bell tower is in relatively poor condition. Both tests gave an average value of velocity of the sonic wave of above 1000 m/s with a uniform distribution among the masonry samples. Visual inspection indeed revealed that the masonry of the belfry suffered from the phenomenon of expulsion of the outer layer of the masonry giving a first hint on the overall condition of the walls. Furthermore, such a low value of wave speed may indicate the presence of voids or, most likely in this case, irregular joint patterns or the inadequate filling of mortar.

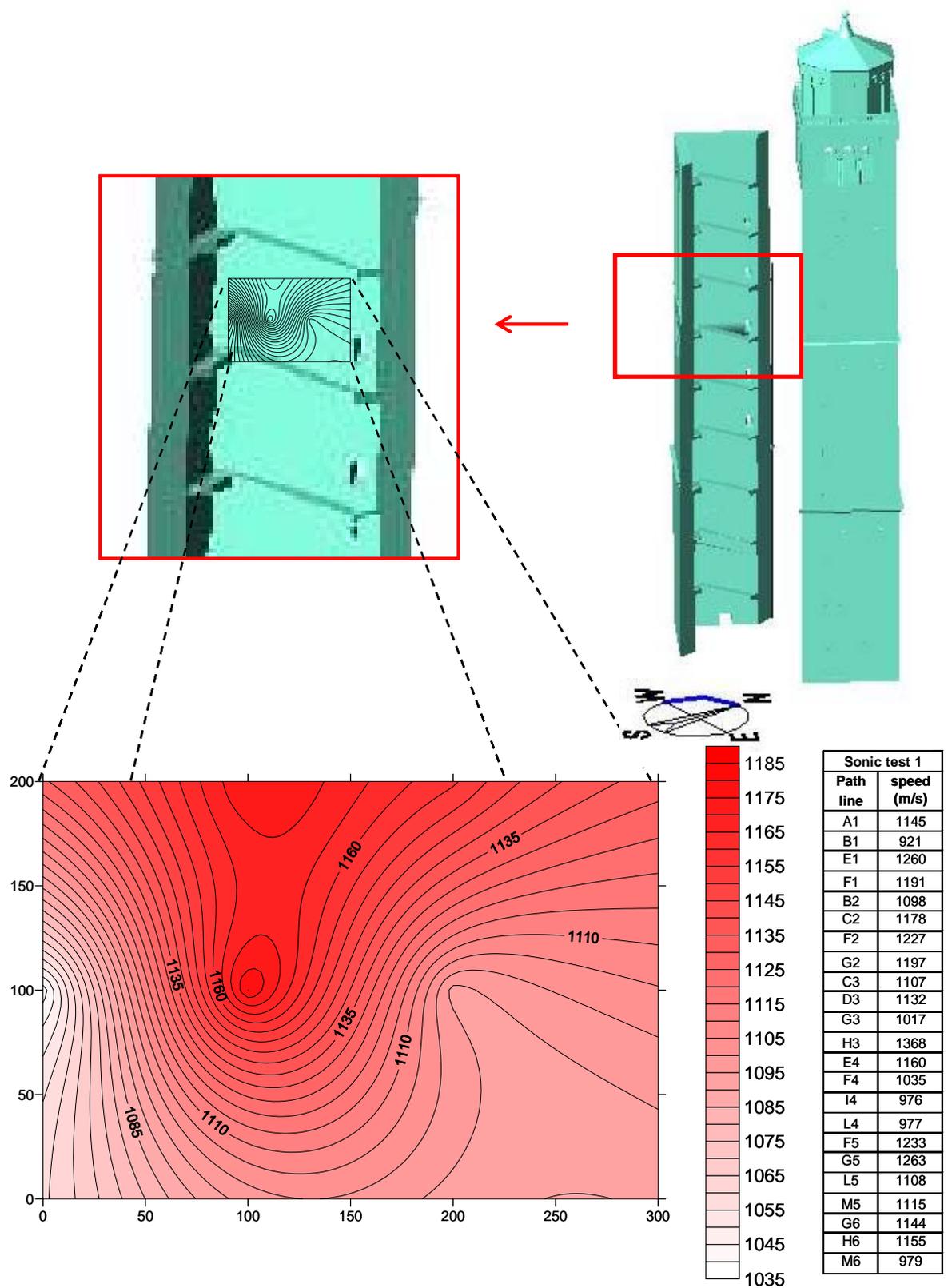


Figure 4-4: Sonic 1, NW wall (unit: m/s)

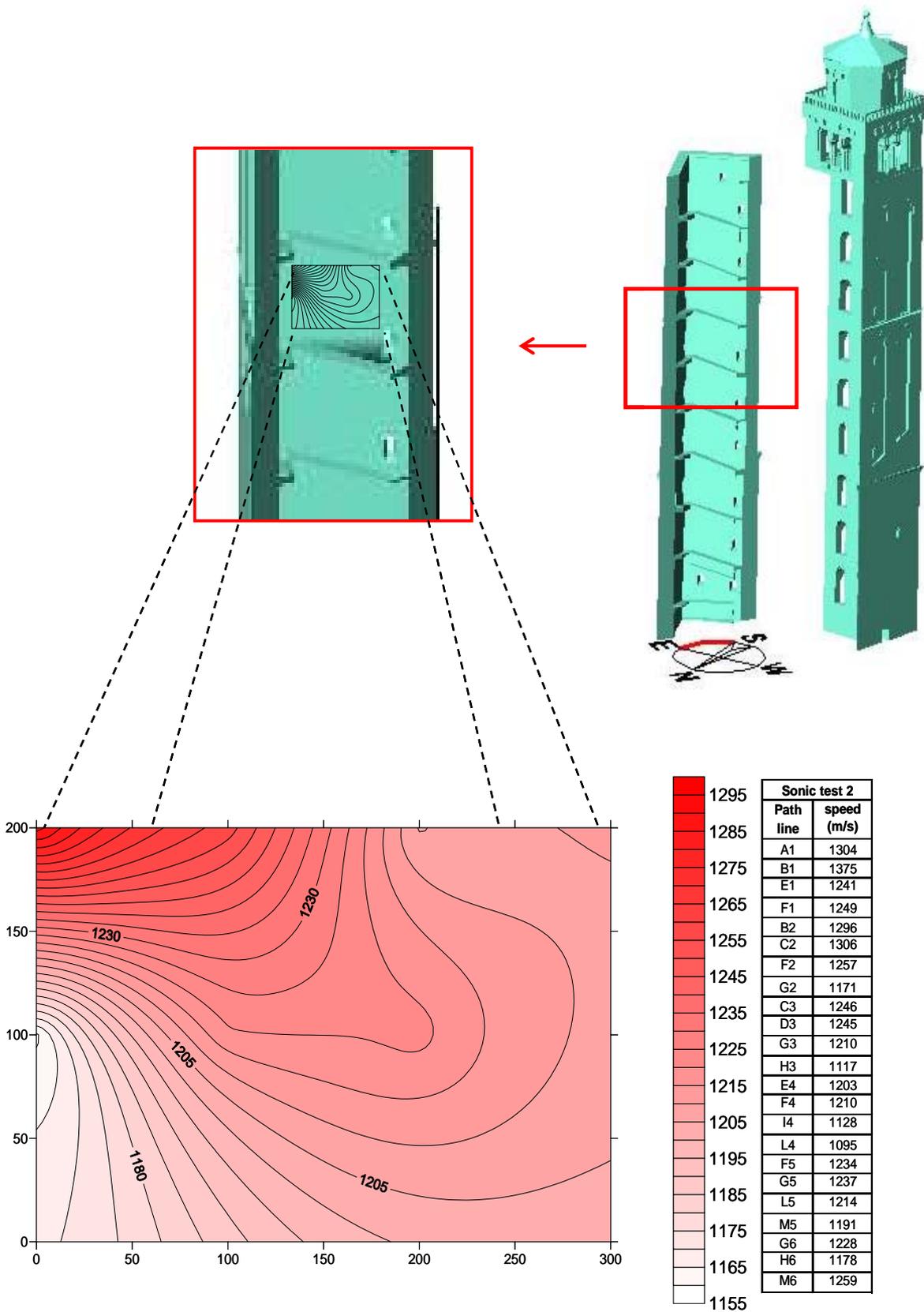


Figure 4-5: Sonic 2, SE wall (unit: m/s)

4.3.2. Core sampling

In order to examine in detail the structural characteristics of a building it is necessary to carry out surveys of small diameter cores. It thus makes it possible to identify the materials and methods of their fabrication and also take samples to be subjected to laboratory tests. This operation becomes indispensable in the very frequent case when masonry consists of two surface layers in regular bond with internal irregular packing. The tests are aimed at determining physical, chemical, mineralogical and mechanical parameters. Areas from where the cores are to be taken must be of adequately hidden position, in order to easily fill the empty spaces left by the samples with the right amount of lime mortar. Moreover the samples must be collected with extreme care in order not to cause unexpected damage around the extraction point. The boreholes can then be used for additional investigations (e.g. video camera survey) which help to define the geometrical properties of the masonry.

A sample from the masonry of the Basilica is depicted in Figure 4-6.

This very sample had been removed from the West corner above the last cross vault of the left aisle. Therefore it is a direct view of the properties of the masonry wall above the deflected column which divides the main nave from the transept.



Figure 4-6: Masonry core sample

At first sight, the collected sample looks to be of pretty good quality: the masonry appears to be in a good conservation state and the mortar seems to be of decent quality and high strength. However, in order to have abundant data to compare, a laboratory test campaign could be carried out to chemically and mechanically describe both bricks and mortar.

4.3.3. Cross hole test

To perform studies on the constitution of the foundation of the bell tower cross-hole test were performed. The cross-hole geophysical surveys consisted of a series of measurements at various depths, of the propagation time of longitudinal and transverse elastic waves between two or more perforations along horizontal paths. With the known distance between the transmitter and receiver, velocities of longitudinal and transverse waves were calculated. Those values were then

represented as diagrams of velocities depending on depth and are strictly related to the characteristics of the investigated elastic material.

More details on this topic can be found in the technical report drawn by **RTeknos s.r.l.**³.

4.3.4. Flat-Jack tests

Flat jack test is a powerful test which allows carrying out direct on site mechanical properties. The test is carried out by introducing a thin flat-jack into the mortar layer of regular and thin joints walls. The test is only slightly destructive hence to be considered in the field of MDTs. Because of these reasons this test has great potential in historical masonry constructions.

Equipment:

- Flat jack: is a thin envelope-like bladder with inlet and outlet ports which can be pressurized with hydraulic oil. Flat jacks may be of any shape in plan, and are designed to be compatible with the masonry being tested.
- Hydraulic system: an electrically or manually operated hydraulic pump with hydraulic hoses is required.
- Displacement measurement: mechanical gauges extensometer measures the distance between fixed gauge points on the wall.
- Gauge points: metal discs or embedded metal inserted as gauge points during the measurement process.

Methodology and results

By doing the **single flat jack** test it is possible to determine the state of stress of the masonry. After cutting a slot in the wall there is a stress relaxation in the wall that causes a relative displacement between the two parts separated by the niche. A thin flat-jack is placed inside the cut and the pressure is gradually increased to obtain the distance measured before the cut. When the relative displacement reaches "0", then the wall is returned to its original configuration and stress state. The explanation of these operations is depicted in Figure 4-7, and Figure 4-8 shows single flat-jack equipment.

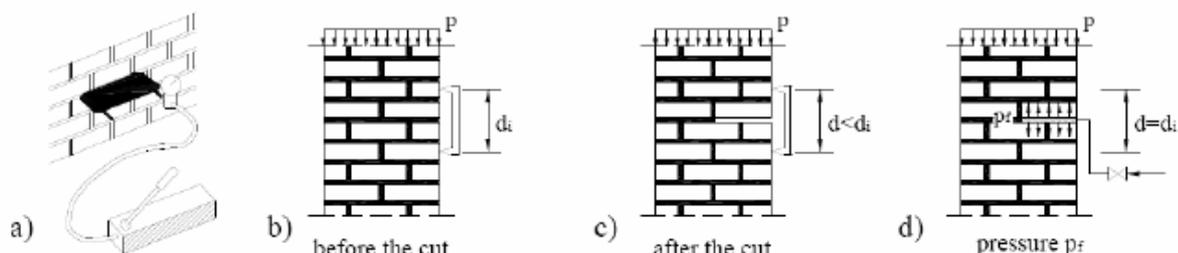


Figure 4-7: Single Flat-Jack operational phases

³ "Indagini diagnostiche sulle fondazioni del campanile della Basilica di S. Maria gloriosa dei Frari, 2ª serie di prove", Technical report, RTeknos, 29/06/06



Figure 4-8: Example of Flat-Jack device

By multiplying the pressure in the flat jack the stress value can be found:

$$\sigma = p \cdot K_j \cdot K_a$$

where:

σ = calculated stress value;

K_j = jack calibration constant;

K_a = slot/jack area constant;

p = flat-jack pressure.

The flat-jack technique can also provide the use of two single flat-jacks placed within a small portion of a wall with a parallel layout. This procedure is called the **double flat jack** test and with this it is possible to determine the elasticity modulus of the masonry (Figure 4-9). The procedure is similar to the single test, only with two flat jacks put parallel. By cutting two parallel slots, part of the wall is isolated from the surrounding masonry forming a "specimen". Masonry between the flat-jacks is assumed to be unstressed and then the wall is loaded in different cycles and the displacement is measured in order to obtain stress-strain diagrams that will give the elastic modulus of the masonry.

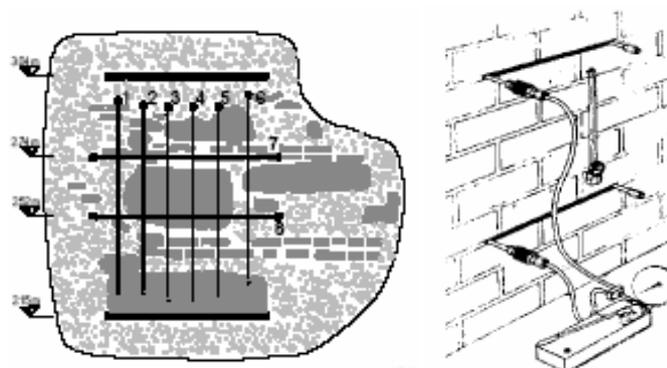


Figure 4-9: Double Flat-Jack operational layout

The flat-jack tests are based on the following assumptions:

- the stress in place of the test is compressive;
- the masonry surrounding the slot is homogenous;
- the masonry deforms symmetrically around the slot;
- the state of stresses in the place of the measurement is uniform;
- the stress applied to masonry by the flat-jack is uniform;
- the value of stresses (compared to compressive strength) allows the masonry to work in an elastic regime.

Particularly, the single flat-jack test is based on the principle of partial stress release and involves the local elimination of stresses, followed by controlled stress compensation.

Every flat-jack test carried out on the Basilica structures was selected depending on the position where test would have been performed or which kind of materials would have been investigated. Depending on which size and dimension, different techniques were used to perform the cut in the wall.

Semicircular cuts where performed on the stone walls by metallic disc fitted with diamond blades and in these slots a flat jack of 350 mm of side and 120 mm of maximum depth. Figure 4-10 depicts a variation of the type of saw used to make the cut.

For rectangular or square flat jacks, cutting is accomplished by running a series of parallel holes by using a drill fitted with a guidance system for the drill-bit. These cuts where performed on masonry walls and the size of such flat jack are 400 x 200 x 3 mm.

Another kind of flat jack had been used in the stone column: in order not to have detrimental effects on the stone, a square flat jack of 150 x 150 mm had been selected for the test within the pillar stones.



Figure 4-10: Metallic disc saw

It should be noted that the disturbance caused by the introduction of the jack in the masonry is very limited as the thickness of the cut is equal to 3 mm. Measurements are carried out using a mechanical removable extensometer that notes the distance between three pairs of *bases* among the cut, made by pairs of thin steel plates pasted on the wall surface.

At the end of the tests it is important that no visible signs of the work remained on the structure. In this case, flat jacks could be easily removed and cuts sealed by lime mortar.

Tests were carried out to determine tensional state in some significant points of the structure. In particular the first campaign performed during the first phase of the work was composed by four single flat-jack tests and four double flat-jack tests in the following positions:

- Single and double test on the NW wall at a height of 6.89 m;
- Single and double test on the SE wall at a height of 9.62 m;
- Single and double test on the NW wall at a height of 41.76 m;
- Single and double test on the SE wall at a height of 40.03 m.

Locations and values of these tests are depicted in Figure 4-11 and Table 4-2.

| Test # | Type of walls | Height location [m] | Young modulus [Mpa] | Stress [Mpa] |
|--------|---------------|---------------------|---------------------|--------------|
| SFJ1 | masonry | 6.89 | | 1.40 |
| DFJ1 | masonry | 6.89 | 5400 | |
| SFJ2 | masonry | 9.62 | | 1.80 |
| DFJ2 | masonry | 9.62 | 2000 | |
| SFJ3 | masonry | 40.03 | | 0.40 |
| DFJ3 | masonry | 40.03 | 2450 | |
| SFJ4 | masonry | 41.76 | | 0.30 |
| DFJ4 | masonry | 41.76 | 3240 | |

Table 4-2: Results of the first campaign

The tests were performed in the opposite SE and NW walls in order to gain a better knowledge of the out-of-plumb of the tower. As it can be inferred from the results, both single flat-jack test pairs #1 and #2 (SFJ1 and SFJ2) and #3 and #4 (SFJ3 and SFJ4) present a common explanation: the higher value within the couples lie in SFJ2 and SFJ3 which confirms that the tower was leaning towards SE, that is, in the opposite direction of the Basilica. A higher compressive state found in those tests ascertain that there was more load on the SE wall or simply that the dead weight was no longer vertical, but the slight eccentricity of the bell tower created a moment force that enhance this behavior.

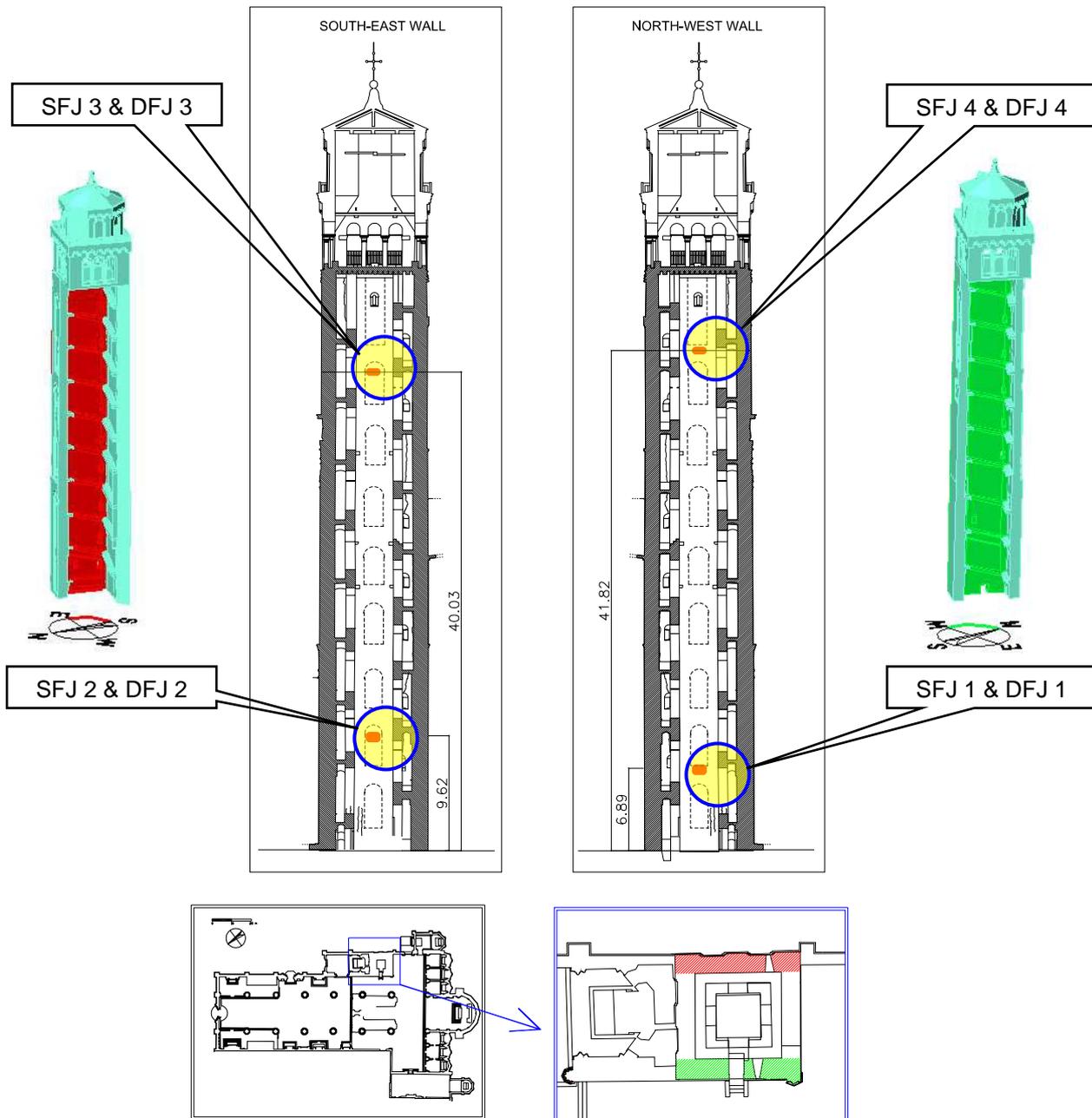


Figure 4-11: First Flat-Jack campaign layout

Different modulus of Elasticity has been found with double flat jack tests (DFJ). In particular DFJ1 and DFJ2 state complete different values among the two walls. Comparing the output stress-strain diagrams of the two tests it is clearly legible that the first has a more elastic behavior than the latter in which, already at the second load cycle, showed a plastic behavior.

Other two test campaigns were performed in the following years by *RTeknos*; the first of these two campaigns comprise:

- No. 3 tests on the outer tower masonry wall, SE side, outside, bottom level;

- No. 2 tests on the outer tower masonry wall, NW side, outside, bottom level, to verify if load eccentricity is present;
- No. 4 tests on the outer masonry wall, SW side, inside, at two different heights among the large crack that diagonally cuts the entire wall in order to check whether the presence of the crack had changed significantly in its tension state;
- No. 3 test on the stem of the stone column between the left aisle and the main nave of the Basilica, immediately below the capital, to verify the stresses transmitted from the stone arch.

These test locations have been defined in agreement with the Landscape and Architectural Heritage Office of Venice. Positions of tests are schematically represented in Figure 4-12.

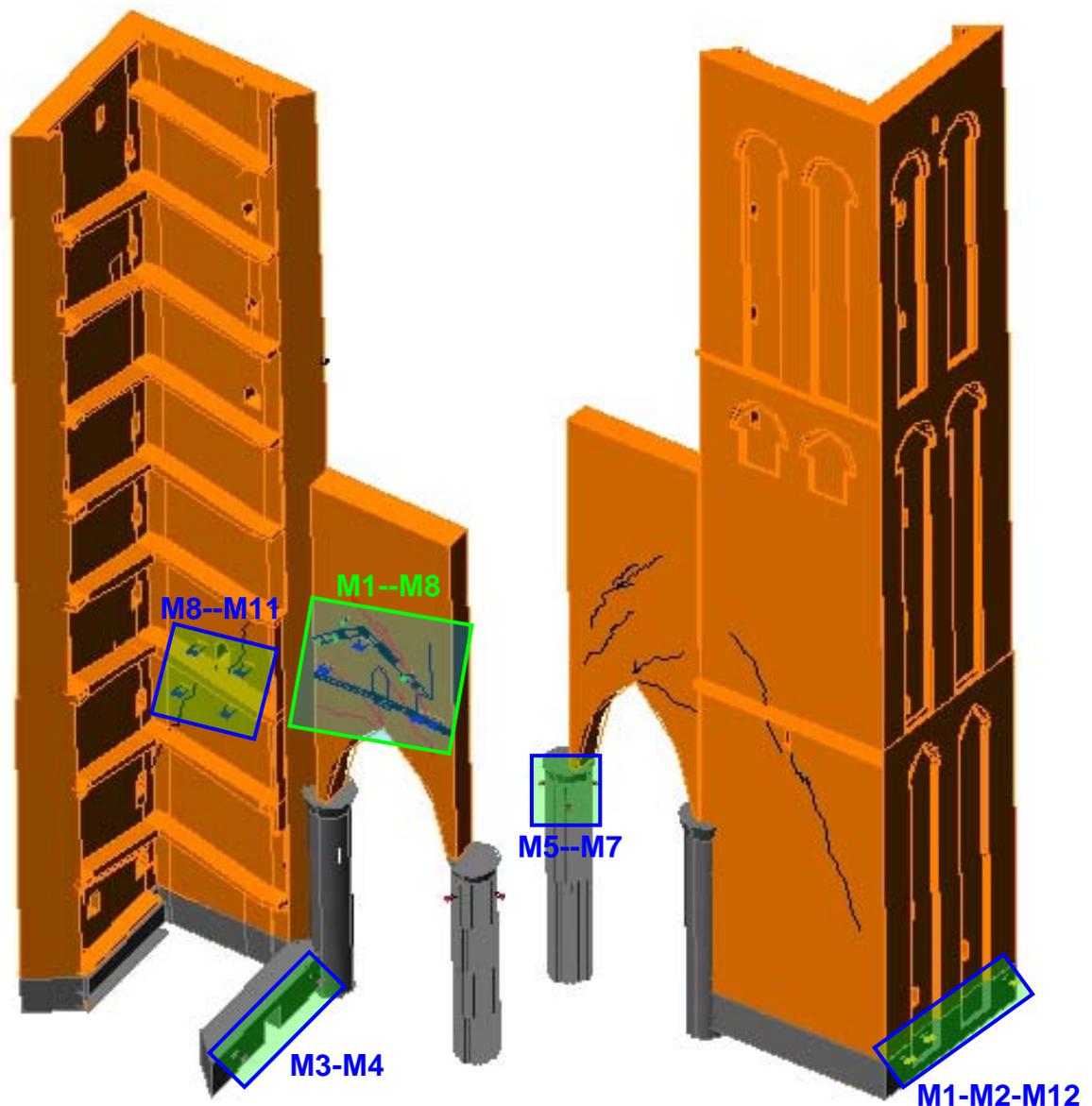


Figure 4-12: Campaigns after 2003 on the Basilica and on the bell tower

In the following Table 4-3 the results of these tests are summarized.

| Test # | Type of walls | Type of Flat jack | Flat jack size | Stress [Mpa] |
|--------|---------------|-------------------|----------------|--------------|
| M1 | masonry | rectangular | 400x200 | 1.80 |
| M2 | masonry | rectangular | 400x200 | 2.25 |
| M3 | stone | semicircular | 325x120 | 1.44 |
| M4 | stone | semicircular | 325x120 | 0.00 |
| M5 | stone | square | 150x150 | 1.76 |
| M6 | stone | square | 150x150 | 3.20 |
| M7 | stone | square | 150x150 | 3.04 |
| M8 | masonry | rectangular | 400x200 | 1.26 |
| M9 | masonry | rectangular | 400x200 | 1.53 |
| M10 | masonry | rectangular | 400x200 | 1.44 |
| M11 | masonry | rectangular | 400x200 | 1.44 |
| M12 | masonry | rectangular | 400x200 | 1.71 |

Table 4-3: Results in brief of the second flat-jack campaign

The first M1 and M2 tests were performed on the SE wall overlooking *Campo dei Frari* (Figure 4-13). Values of these two tests highlighted a different distribution of stresses along the South-east wall, most likely cause of the out-of-plumb of the tower. Stress value of 2.25 MPa was recorded in the East corner (M2), higher than the one recorded at the South angle (M1 = 1.8 MPa).

These tests were performed on the pilasters present in the corners. Since the pilasters were affected by local phenomenon of bulging it was necessary to carry out an extra test (M12) just beside them. Therefore the test quoted a value very close to the one from the test next to it (1.71 MPa) confirming that the bulging phenomenon did not actually affect the previous test and its value was reliable.



Figure 4-13: Single Rectangular Flat-Jack test M1, M2 and M12

Two other tests were performed in the tower wall inside the Basilica, M3 and M4. Both tests were performed at about 1.5 m from the ground, where the stone basement is still present. Experts decided that semicircular flat-jacks would have been suitable for this task (Figure 4-14).

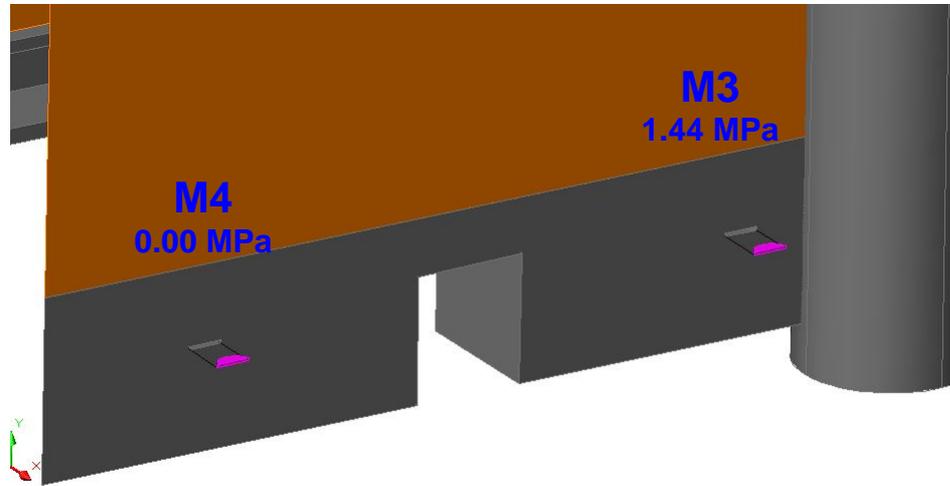


Figure 4-14: Single Semicircular Flat-Jack test M3 and M4

These tests emphasized another kind of behavior: M4 stated that the stone is unstressed, while M3 stated 1.44 MPa compressive strength. This could mean that, if the test has been correctly carried out, no stresses are transmitted in that portion of the wall, thus confirming an out-of-plumb of the bell tower. The same phenomenon can be explained by defining the composition of the wall in those areas: large stone blocks that may transfer vertical load in a non homogeneous way, but only through the areas of contact between the irregular surfaces of the stone blocks.

Therefore, the average compressive stress measured at the base of the tower was estimated at 1.92 MPa in the outer side and 1.44 MPa in the inner side of the Basilica. The lower compressive state found in the inner side of the Basilica was most likely due to the fact that within the Basilica, part of the dead load of the tower is transmitted to the column by the arch stone (Figure 4-15).

Test from M1 to M4, (including test M12) were mainly performed to determine the importance of the out of plumb of the bell tower. These test would have validate the results of the geometrical survey and the precision altimetric survey, in addition confirmed the outcome of the previous campaign.

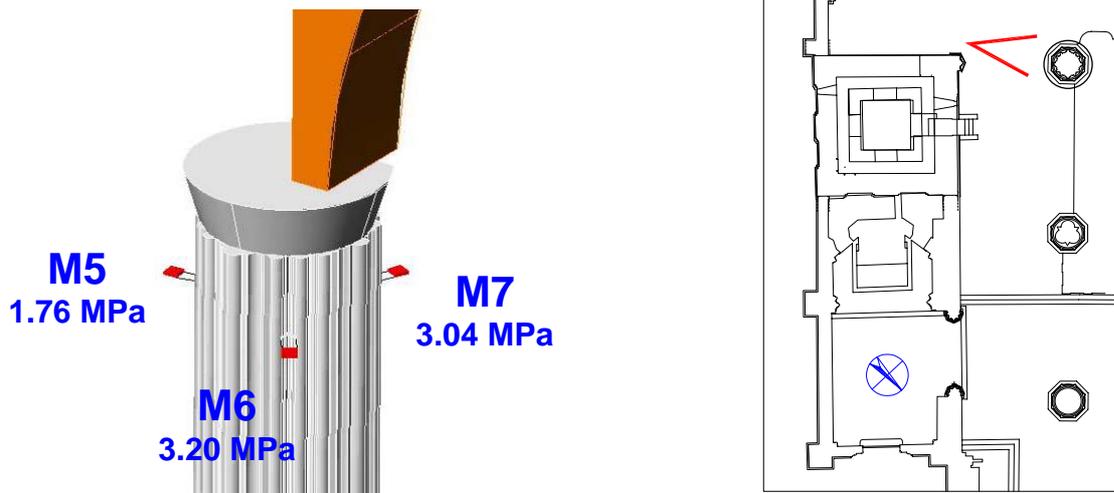


Figure 4-15: Single Square Flat-Jack test M5, M6 and M7

To confirm this theory, additional tests were carried out on the upper part of abovementioned column. Three tests were performed on the surface column shaft, just below the capital, arranged in three directions, 120° apart from each other (Figure 4-15). Not only was a higher load on the column found, but this load came with a high eccentricity that produced a bending moment on top of the column. The column tended to have a differential shift towards the main nave due to the eccentric horizontal load coming from the bell tower. Evidence of this phenomenon was given by the discovered values. Tests showed an uneven distribution of compressive stress, with a load concentration in the part of the column facing the tower. In this area, test M6 and M7 registered stress values of 3.20 and 3.04 MPa, in strong opposition with 1.76 MPa value registered by M5 test diametrically opposed.

This flat jack campaign ends with the need to understand how the portions of the transept wall among the crack behave (Figure 4-16).

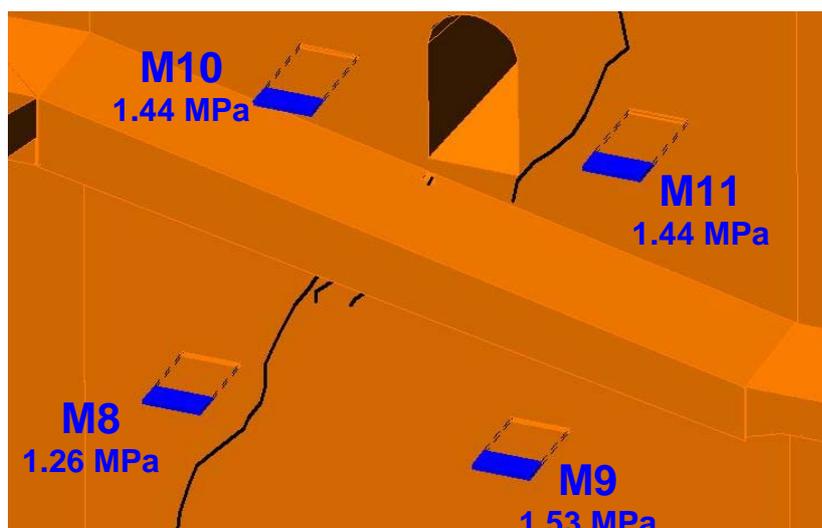


Figure 4-16: Single Rectangular Flat-Jack test M8, M9, M10 and M11

These tests were carried out among the two parts of the SW wall of the tower separated by the main oblique crack. Tests M10 and M11, performed close to the window facing the transept of the Basilica about 15 m high, show the same stress load between the two parts among the large diagonal crack (1.44 MPa). In a lower level (approximately 3 m below) M8 and M9 tests showed an uneven distribution of stress with a value of 1.53 MPa (the one closest to the relieving arch) and 1.26 MPa (opposite side). It is difficult to explain this stress distribution but the main reason can lie in the fact that M10 and M11 were carried out in a portion of the wall in which the crack may have been less deep through the thickness of the wall, thus no variation of loads can arise. On the other hand, the slightly different values recorded between test M8 and M9 can be linked to the more influence of the crack which can lead to separation or shift between the two portion of the walls. However, the average value between test M8 and M9 is 1.41 MPa which is very close to the value measured approximately 3 m above.

Being known the overall behavior of the tower as a matter of stress distribution related to the values of the second group of flat jack tests, the experts suggested to investigate the real behavior of the relieving arch on the transept wall, by designing a third flat jack campaign around this area of the structure. In the following Table 4-4 the results of these tests are summarized and the locations are depicted in Figure 4-17.

| Test # | Type of walls | Type of Flat jack | Flat jack size | Stress [Mpa] |
|--------|---------------|-------------------|----------------|--------------|
| M1 | masonry | rectangular | 400x200 | 0.00 |
| M2 | masonry | rectangular | 400x200 | 0.09 |
| M3 | stone | rectangular | 400x200 | 0.63 |
| M4 | stone | rectangular | 400x200 | 0.95 |
| M5 | stone | square | 150x150 | 0.00 |
| M6 | stone | square | 150x150 | 0.72 |
| M7 | stone | square | 150x150 | 0.00 |
| M8 | masonry | square | 150x150 | 0.56 |

Table 4-4: Results in brief of the second flat-jack campaign

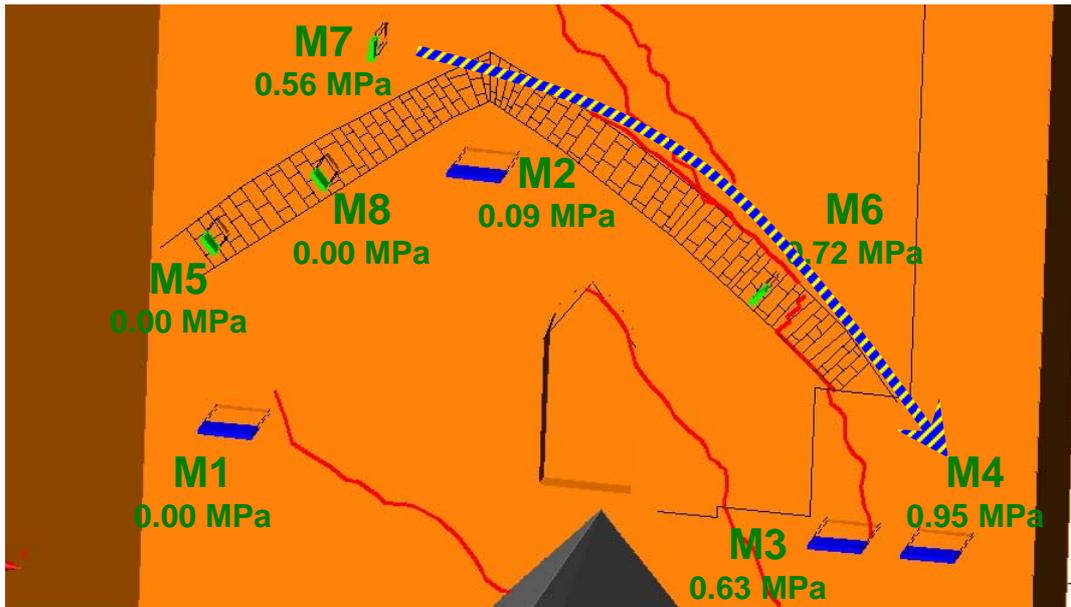


Figure 4-17: Summary of compressive state measured by the third single Flat-Jack campaign

Tests M1, M2, M3 and M4 were carried out with rectangular flat-jacks (blue in Figure 4-17) concerning the masonry wall, while tests M5, M6, M7 and M8 were performed with smaller square flat-jacks (green in Figure 4-17) involving the relieving arch. Each test was made for a purpose, in particular:

- Test M1 to detect compressive stress to be compared with test M3 and M4;
- Test M2 to check the real behavior of the relieving arch above it;
- Tests M3 and M4 to detect stress differences between the two portions of the wall along the crack;
- Test M5, M6, and M8 to prove the efficiency of the masonry relieving arch;
- Test M7 to detect the horizontal stress, hence the load the tower is transferring to the Basilica.

These latest test results clearly indicated the formation of an “arch behavior” through which the stresses were transferred from the tower to the investigated wall. This arch behavior particularly affected test points M7, M6, M4 where the stress values have been measured between 0.56 and 0.95 MPa. In other test points, outside this imaginary arch, the stresses were null or negligible.

As the ones from the previous campaign, these results clearly indicate the flow line through which the load of the tower was transmitted to the adjacent column (as depicted with the dashed line in the same Figure).

Regarding the effect of the large crack on the SW tower-transept wall, it can be observed that tensional states measured on two distinct levels and on opposed side of the wall are all very similar, which indicates that the fracture, although running through the entire thickness of the masonry wall, does not induce disruption of appreciable size on load distribution along this side of the tower.

4.3.5. Strain gauges on metal tie rods

After one year from the installation of the monitoring system, it became necessary to check whether the annual cycle of thermal variation caused new strain variations on metal tie rods installed in the bell tower. A mild loss of tension had been registered. The first tie rod to be investigated suffered a permanent deformation of approximately $-80 \mu\epsilon$ while the second tie rod $-25 \mu\epsilon$. This corresponded to a loss of tension on both tie rods which were: 24 kN and 7.5 kN respectively. These values come from the relation between the Young modulus of the tie rod and the measured strain (Table 4-5):

$$\sigma = E \cdot \epsilon = 210000 \cdot 1 \times 10^{-6} = 0.21 \text{ Mpa}$$

| | |
|----------------|-------------------|
| E = 210000 Mpa | |
| Tie rod #1: | |
| $\epsilon =$ | -80 μm |
| $\sigma =$ | 16.8 MPa |
| d = | 43 mm |
| Load = | 24.40 kN |
| Tie rod #2: | |
| $\epsilon =$ | -25 μm |
| $\sigma =$ | 5.25 MPa |
| d = | 43 mm |
| Load = | 7.62 kN |

Table 4-5: Measured load on metal tie rods

The monitoring of these measurements needed to be updated with new records for the following years as a one year period time is such a short period to clearly evaluate the effects of irregular daily thermal variations.

4.3.6. Direct Pendulum

A certain amount of information for soil consolidation was obtained from the measurements provided in real time by direct pendulum combined with the results of the precision topographic survey. Even the EL1 and EL2 gauges, installed on the ashlar of the arch connecting the tower to column of the Basilica, gave valuable information on the conditions of interaction between church and bell tower during the intervention works. Both results from the monitoring of the pendulum and the crack meters are discussed.

The results of the direct Pendulum are depicted in the following Figure 4-18. This image show the period from 17th December 2003 (date of installation of the monitoring system) to 30th August 2007. The yellow highlighted period is from July 2005 and March 2006 while the execution of the soil works occurred (boreholes, tests, injections). Injections were carried out strictly from September 2005 to January 2006, but it can be seen from this graph that their influence concerns a longer period.

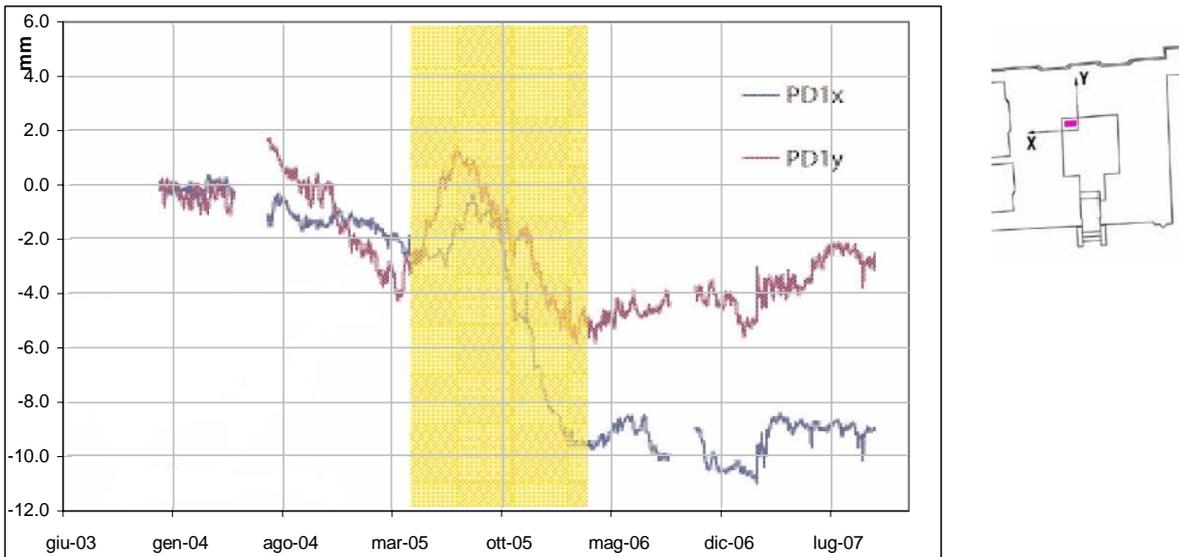


Figure 4-18: Direct pendulum measures

The trend of measurements related to the X component of the displacement of the direct pendulum, shown as PDx, was observed during the period December 2003 and April 2005: a shift of the tower towards the apse with fluctuations related to seasonal thermal changes is noticed; however the displacement is not more than 2 mm. In this period, the velocity of movement of the X component of the pendulum can be estimated as approximately 1.5 mm / year.

| | |
|-------------------|---------|
| speed = 1.48 | mm/year |
| crack width = 2.1 | mm |
| time = 518 | days |
| time = 1.42 | year |

There was however rapid increases in the displacement of the tower towards the apse of the Basilica from July 2005 until March 2006. This was due to the work of soil fracturing for the consolidation of the soil beneath the tower.

The shift measured on top of the tower was approximately 9 mm during the period mentioned above. After the completion of the work, the speed of deformation decreased significantly and between March 2006 and January 2007 showing a trend similar to the one found during the previous period.

In the latter period, leading up to November 2007, a further slight shift of the tower was observed towards the apse with values not exceeding 1 mm, which most likely developed in relation to seasonal thermal variations.

In January 2007 a maximum value of a 10 mm shift towards the apse was reached. It is relevant to mention that during the period January 2007- August 2007, there was an important slowdown of the shift of the tower and therefore in the speed of X component. With a further closer look at the graph, eliminating as much as possible thermal effect variations, the tilting speed assumes values

less than 0.5 mm/year. This significant reduction of the shifting speed of the pendulum, if confirmed by future long term observations, seems to indicate that the consolidation of the soil with the technique of soil fracturing had achieved positive results.

On the other hand, measurements of the Y movement component, indicated with PD_y, were observed during the period between December 2003 and April 2005. At this stage, with respect to the seasonal thermal variation of the X direction, the values are higher and ranged between 2 and 4 mm. During the period of work (between April 2005 and March 2006), there was a shift of the bell tower towards the interior of the Basilica amounting to about 5 mm. From March 2006 until August 2007 there was a slight, gradual shift of the tower towards "*Campo dei Frari*" (about 3 mm) with modest variations linked to thermal seasonal changes.

4.3.7. Crack meters on the stone arch

Gauge EL1 (Figure 4-19) recorded a rapid increase of the opening of the crack of about 0.4 mm from November 2004 to mid February 2005 (Figure 4-20, yellow) with deformation speed of approximately 1.17 mm per month.



Figure 4-19: Gauge EL1

| | |
|-------------------|----------|
| speed = 0.11 | mm/month |
| crack width = 0.4 | mm |
| time = 105 | days |
| time = 3.50 | month |

Such increase, occurred during winter time, seems not only related to thermal variations, as long as comparing the trends during the same period of the previous year (Figure 4-20, blue). Without considering thermal effects (very regular after all), the permanent opening increase was about 0.25 mm, by simply comparing the value recorded in the same month of January in 2004 and 2005.

A very small deformation (0.04 mm) can be the result of a light earthquake in July 2004.

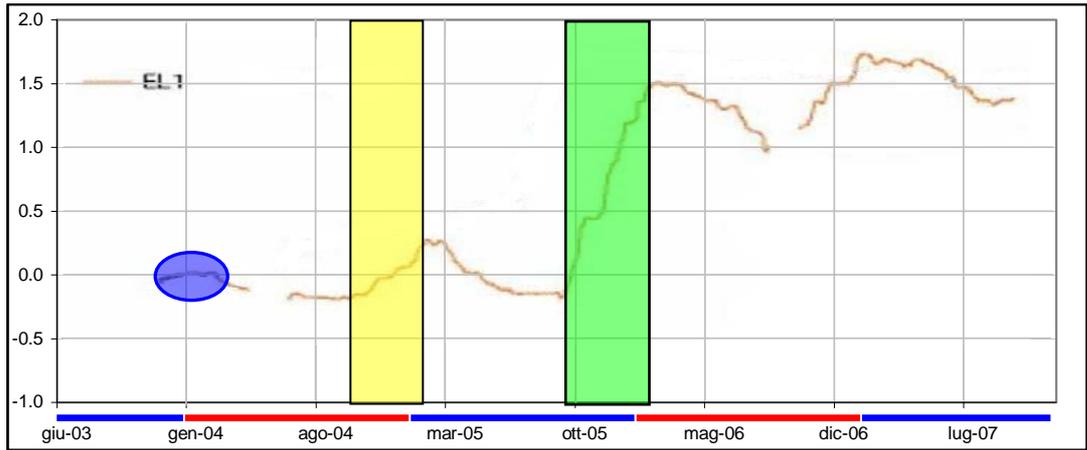


Figure 4-20: Crack meter EL1

EL1 recorded another rapid opening movement of the crack during September 2005 and January 2006 (green Figure 4-20). Those values (and related movements) are undoubtedly associated with the consolidation works of the soil and they are higher than the measurements correlated with thermal variations (which are of 0.5 mm as indicated by the diagram of openings detected after January 2006).

Hence the openings of the cracks are related, and happened simultaneously, to the movements of the bell tower which, in the same period, suffered an inward movement towards the Basilica, thus compressing the arch that separates it from the adjacent column.

The wooden props for the stone arch revealed themselves to be very useful.

Crack meter EL2 did not seem to be affected by thermal variations as its record indicates a smoother path compared with the results of crack meter EL1. As a matter of fact, this trend is clearly related to its position: despite EL1, crack meter EL2 is in a vertical direction and it was placed among the stone arch and the masonry wall. (Figure 4-21).

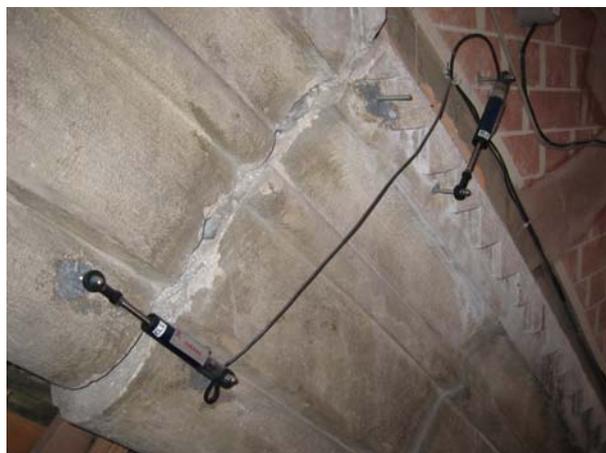


Figure 4-21: Position of gauges EL1 and EL2

EL2 showed a progressive opening up to about 0.6 mm during the period from December 2004 to July 2005 (yellow in Figure 4-20) with a sudden opening of the crack of approximately 0.2 mm during the earthquake in July 2004. After that, another rapid increase equal to 1.2 mm occurred during the period from July to October 2005 (cyan in Figure 4-20). After that date a slight closure is observed (equivalent to about 0.3 mm), with constant evolution.

The path of EL2 crack meter (Figure 4-22) can be divided in three separate sections, each of them with the following speed per month characterization:



Figure 4-22: Gauge EL2

| | | |
|--|--|--|
| speed = 0.03 mm/month crack width = 0.6 mm time = 555 days | speed = 0.30 mm/month crack width = 1.2 mm time = 120 days | speed = 0.01 mm/month crack width = 0.3 mm time = 750 days |
|--|--|--|

By comparison of the widths recorded in December 2003 (0.05 mm) and the one recorded in September 2007 (1.55 mm) it can be stated that a significant deflection of a permanent nature, not linked to thermal variations, is about 1.5 mm. Subtracting the value of deformation caused by the earthquake (0.2 mm), the final permanent width of the crack is 1.3 mm in about forty-six months (Figure 4-23).

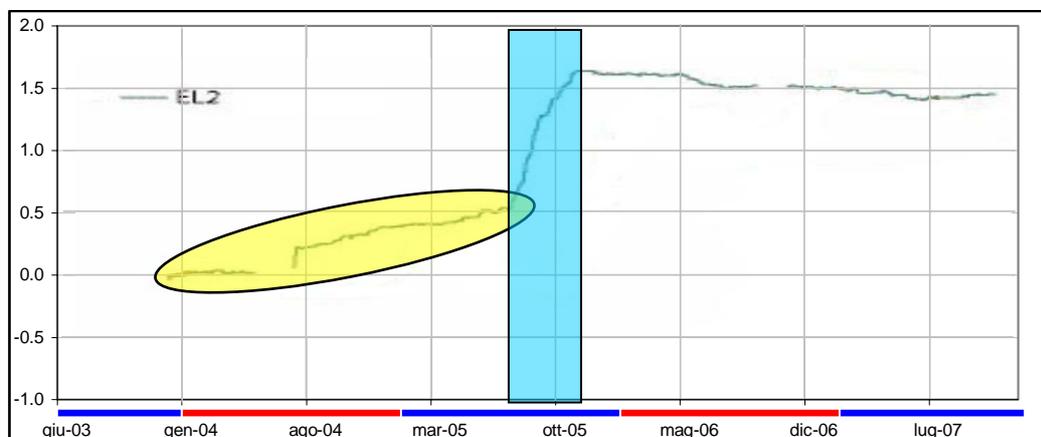


Figure 4-23: Crack meter EL2

4.3.8. Precision topographical survey

To better understand the position of the benchmarks of the topographical survey carried out throughout the first phase of the work on the *Basilica dei Frari*, Figure 4-24 and Figure 4-25 shows a 3D view of the portion of the church that concerns the survey. The benchmarks are highlighted with a red spot and corresponding number.

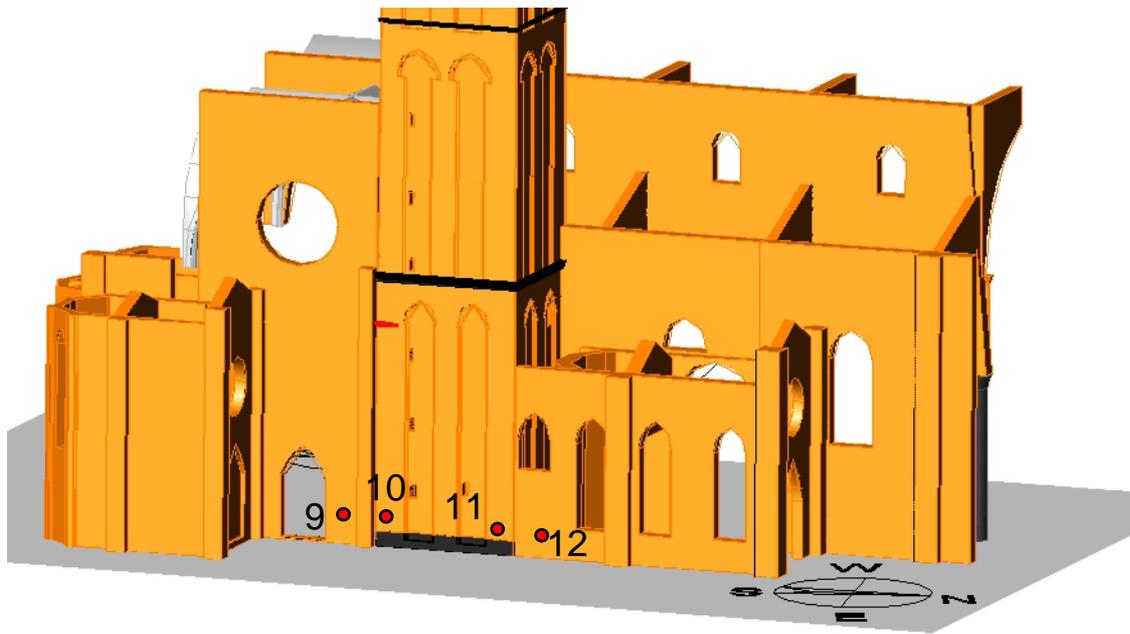


Figure 4-24: Benchmarks of the precision topographical survey – SE view

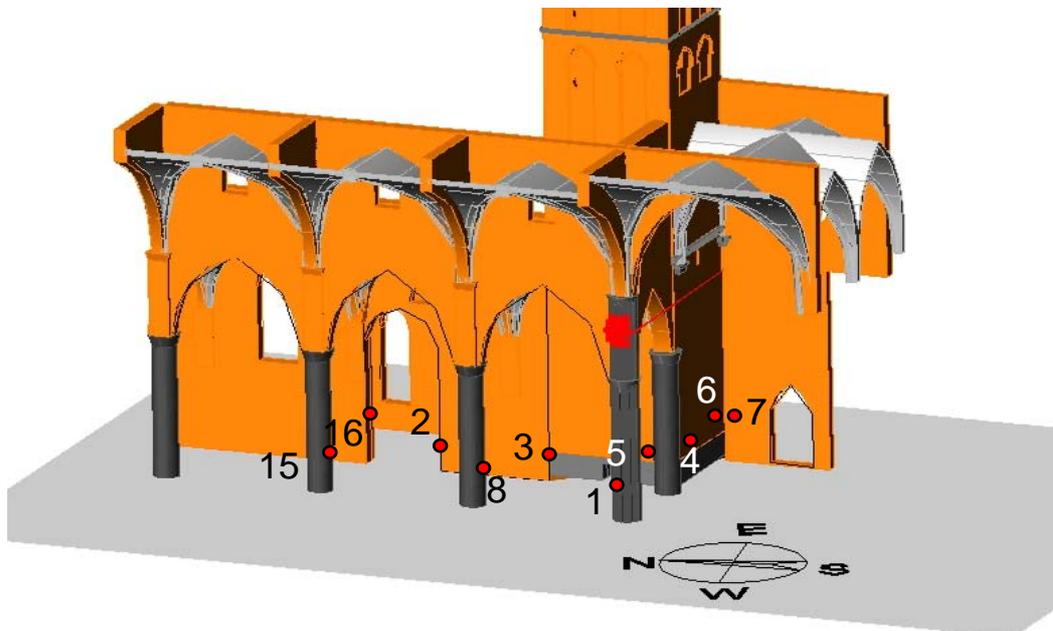


Figure 4-25: Benchmarks of the precision topographical survey – NW view

The most significant altimetric changes can be noticed between April 2005 and March 2006, while analyzing data of the benchmarks on the Basilica and on the tower walls. Figure 4-26 shows the settlement of the various points of the tower between 31st March 2005 and 27th March 2007, weekly monitored.

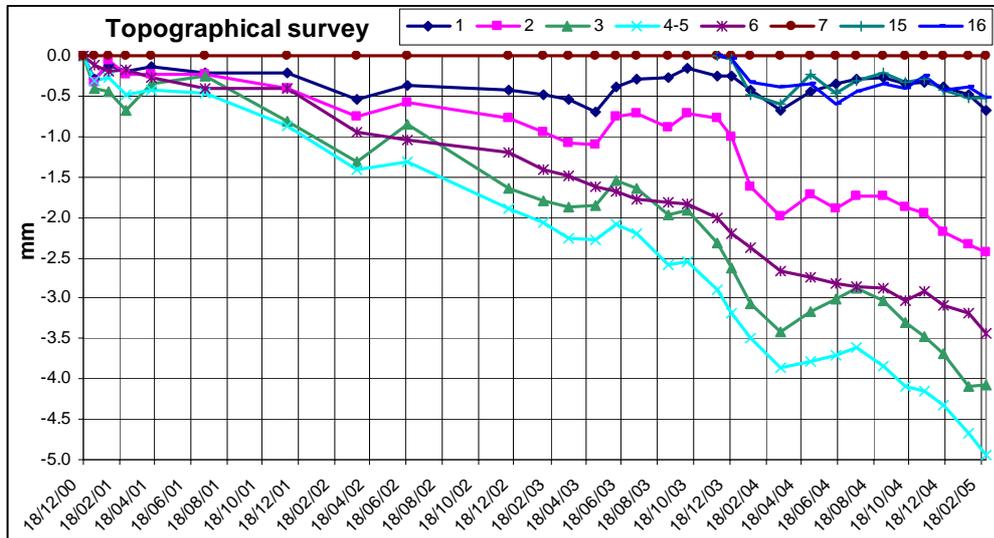


Figure 4-26: Topographical survey from 18/12/00 to 24/02/05

Benchmarks from #1 to #7 were installed on 18/12/00 while benchmarks #15 and #16 were installed later on 26/11/2003.

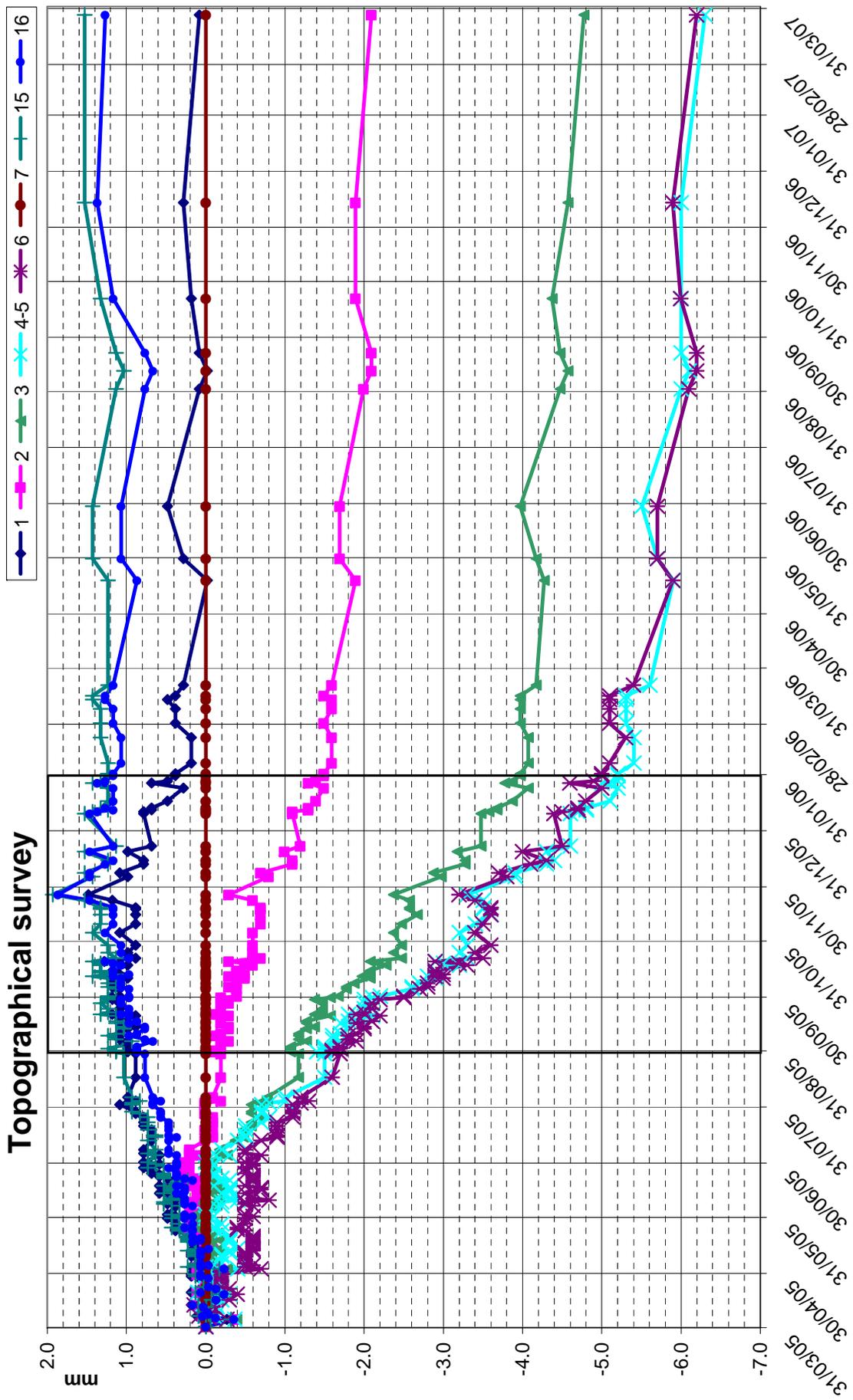


Figure 4-27: Data from the topographical survey

The yellow section in Figure 4-27 represents the period during which the work on the soil foundation occurred. The maximum reduction was always registered in points #4 and #5, located on the edge, toward the apse. The values of this settlement can be divided as follows and represent the periods of the start, the duration, and end of the work (Figure 4-28, Figure 4-29 and Figure 4-30):

Period from 18/12/2000 to 01/09/2005: - 1.50 mm

Period from 01/09/2005 to 31/01/2006: - 5.20 mm

Period from 31/01/2006 to 27/03/2007: - 6.30 mm

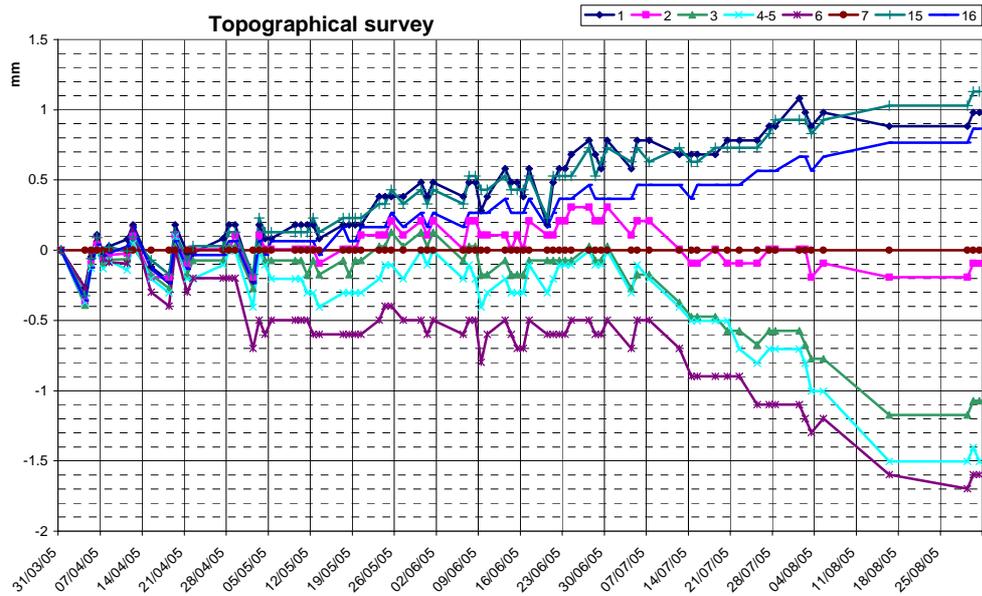


Figure 4-28: Settlements before execution of the work

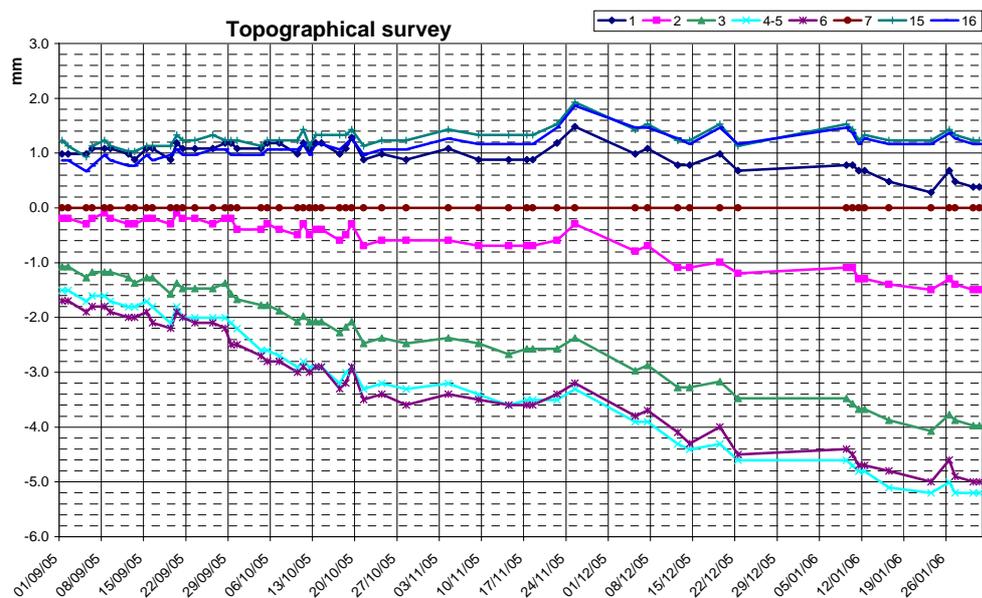


Figure 4-29: Settlements during execution of the work

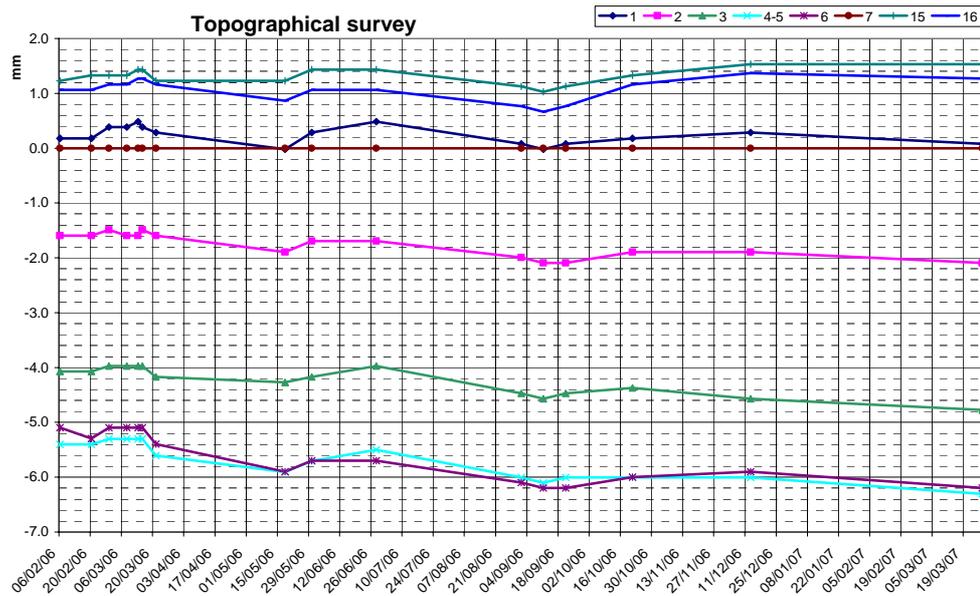


Figure 4-30: Settlements after execution of the work

A closer examination of Figure 4-27 allows the reader to notice the trend of the rotation of the tower towards the interior of the Basilica. Thanks to these measurements it is also clear that the phenomenon of differential settlements affects only a small portion of the Basilica as the survey clearly stated a negligible value (close to zero) of benchmark #1 placed on the pillars of the main nave. Furthermore, benchmarks #15 and #16, placed respectively on a pillar far from the tower and on the door of St. Peter's Chapel, show positives value, hence an upward movement.

During the same period, benchmarks #3-4 and #10-11, placed in opposite directions, visibly marked the rotational trend of the tower outwards from the apse. Before any other consideration, the following unusual behavior must be noted: during the half year period that runs from July 2004 and January 2005, the same couple of benchmarks recorded the trend to rotate in opposite direction; the first couple (#3-4) indicated a rotation towards the front façade, while the latter (#10-11) towards the apse.

All the graphs of the settlements related to each benchmark are reported in Appendix B.

The strong variations clearly visible in Figure 4-29 are without any doubt linked to the work of soil fracturing and are consistent with information provided by the direct pendulum. The maximum settlement was registered in benchmarks #4 and #5 located in the corner of the tower towards the apse, with maximum value of -11.94 mm. The next examination of the settlements in other cornerstones #3 and #10, respectively -9.28 and -8.59 mm, indicated that the direction of movement of the bell is facing the apse and towards the nave Figure 4-31. The same Figure illustrates the trend of settlements by means of vectors (arrows point toward the bigger settlement) and colors. In the latter the behavior of the surveyed structure is clearly visible: the largest settlement is around benchmarks #4 and #5 (blue zone), then the settlement decreases while moving out from the tower (red zones).

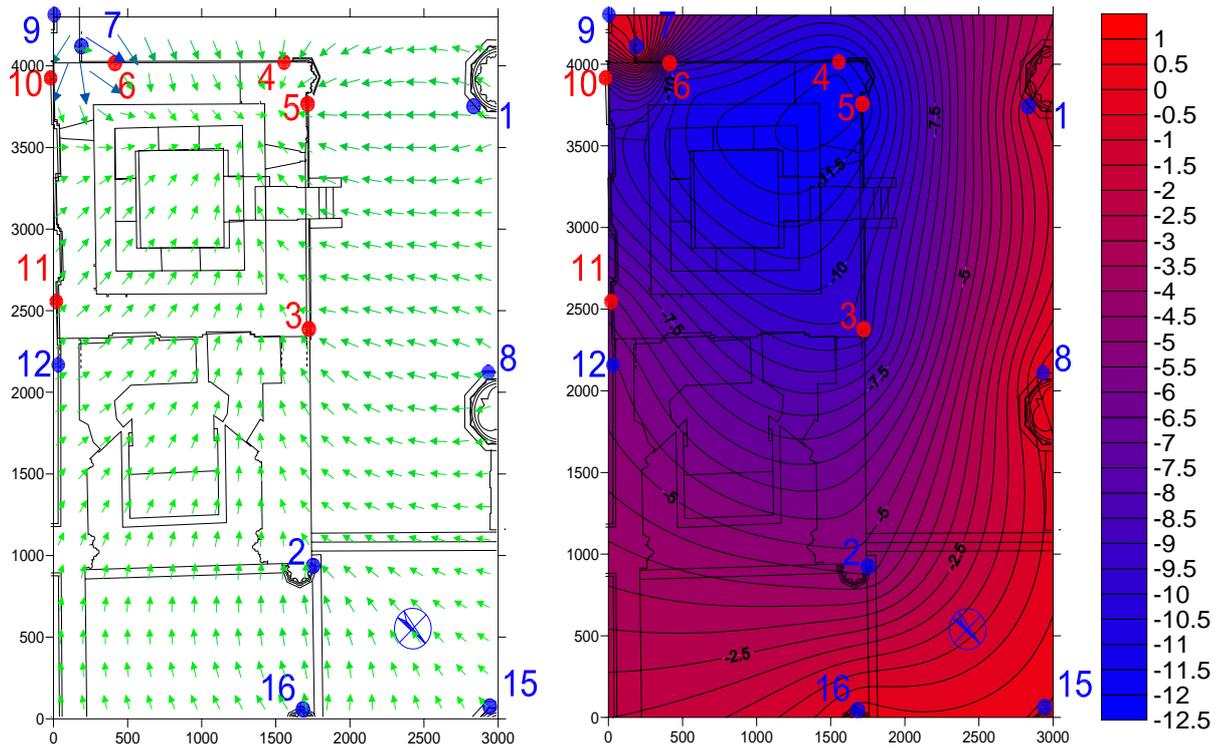


Figure 4-31: Settlement pattern [legend in mm]

The same information is even better visible in the 3D image in Figure 4-32 representing the final values of the settlements measured on 27th March 2007.

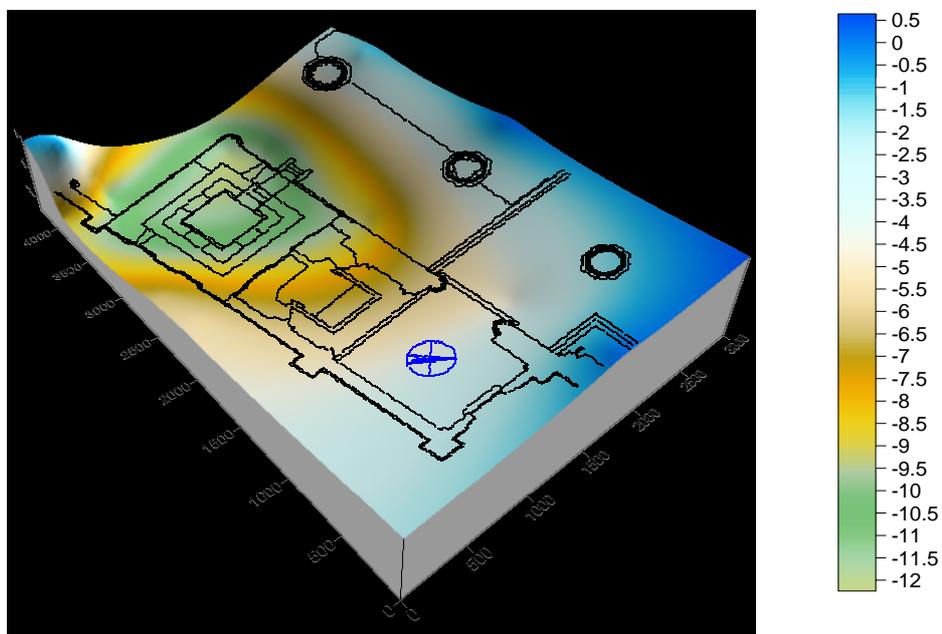


Figure 4-32: 3D image representing the settlements [legend in mm]

It was the inclined crack pattern above the ogival arch (along the wall of the transept) that revealed the essential aspects of the ongoing mechanisms. The crack pattern followed the trend of the

isostatics of traction in the portions of the wall that connect tower and Basilica and in portions where the stress exceeded the resistance of the material. This gives evidence of inclined struts (portions of masonry between the cracks) capable of transmitting relevant components of vertical and horizontal forces from the bell tower to the church. Therefore the out-of-plumb played a negative role on the conditions of stability of the tower. The leaning increases the eccentricity of the vertical load (weight of the tower, with relatively high center of gravity), hence increasing differential settlements within the base of the tower.

Finally, through the inclined sampling cores of the soil, it was possible to evaluate a differential settlement of approximately 17 cm between the part of foundation beside "*Campo dei Frari*" and the original foundation towards the SE direction, in line with the current inclination of the tower of 0.90°.

4.3.9. Other tests to be proposed

The results obtained thus far with the proposed monitoring system, NDTs and MDTs are sufficient enough to clearly explain the behavior of the Basilica and to help calibrate a finite element model. However, different kinds of other assessments would have been performed to locally define particular characteristics of the building. The following are some of the tests that have not been taken into account and are briefly discussed, together with their purposes.

Radar investigation

The radar testing technique is non destructive test that uses high-frequency electromagnetic waves emitted through an antenna with very short impulses and allows determining location of separation surface between materials with different dielectric constants. These tests could have been performed to locate confined anomalies in the masonry (such as voids) or where important wall paintings cannot be touched or removed.

Rebound test

The rebound test is aimed at a qualitative evaluation of the compressive strength of mortar and/or superficial strength of the stones or brick material. It can also provide information on local damage of the material. Of course this technique has been outdated by the flat jack tests with its more reliable information but it could have been a simpler and quicker way to determine, at the beginning, the strength of the material.

Magnetometric analysis

Magnetometry locates the presence of metallic elements within the masonry structure. This technique could be important in determining position of internal metallic elements that could undergo the oxidation phenomenon, increasing their volume and therefore creating dangerous stress states within the masonry.

Thermo-graphic analysis

The Thermo-graphic analysis is based on the thermal conductivity of the material. This as well is a non destructive test and is used mostly in those cases where the object to be investigated is covered by plaster or frescos and hides construction anomalies. This would be useful to investigate the masonry of the Basilica covered by Regalzier paintings.

Dynamic identification

Interesting, especially for the bell tower, would have been carrying out a dynamic identification to determine, among other things, the mode shapes and frequencies. This information could help further calibration of the finite element model.

Pyloclin and Resistograph

Besides the fact that during recent interventions and reinforcement campaigns, wooden structures of roofs and vaults did not emphasize any sort of problems, it would be interesting to carry out Pyloclin and Resistograph test. These tests belong to the category of non destructive tests. The first consists of transforming the elastic potential energy into impact energy, to correlate the density and elastic properties with the depth reached with the needle of the device, while the latter measures the strength of the timber by measuring the resistance of the drill-bit while twisting through the timber specimen.

5. STRENGTHENING TECHNIQUES

5.1. *Acquiring informations*

So far it has been discussed the damage of the structure and the results from the survey that have been conducted.

From these, it is possible to state what are the main structural problems related to the construction work that led to the considerable differential settlements between the bell tower and the Basilica. Some of the cracks related to these differential settlements are of minor importance and can be easily repaired (this is the case of many historical buildings in which most of the time it is sufficient to fill a crack or locally repair part of the pavement to solve the problem). Some other cracks on the other hand do not show this easily reading of their opening history and may lead to the formation of mechanisms between adjoining structures. Some examples of the first typology are the interventions taken in St. Peter chapel and the work carried out to hide the evident deflection on top of the stone arch connecting the bell tower with the nave at the transept intersection. Some of the latter typology, much more complex and insidious, will try to be explained in this chapter.

After accurate analysis of the results from the in situ tests and the monitoring system it can be stated that the consequences of the changed path of the loads and the sinking of the bell tower can be attributed to:

- an increase in the compressional state and the birth of a flexural moment are noticed in the column that separates the transept from the main nave; this increase in compression is compared with the normal static behavior that would have affected the column if the bell tower would have been independent. The flexural moment is related to the increasing horizontal thrust from the tower and the eccentric vertical load.
- a decrease of vertical load of the bell tower which is certainly related to the migration of part of the load to the adjoining column, and of course a flexural bending moment (of the same entity of the one affecting the column, but less important considering the size of the tower); the main crack in the SW wall is related to these phenomenon.

5.2. *Renovation works*

Since the fall of the bell tower of St. Mark, the worrying conditions of all the towers in Venice were investigated. The intervention work performed to the complex of Frari during last centuries can be summarized as follows:

5.2.1. XIX centuries renovation works

Well documented are the numerous nineteenth century interventions, which tried remedy the deterioration in various parts of the Basilica, in particular the static problems mainly affecting the area where the Basilica is contiguous to the bell tower.

The first documented intervention on the bell tower, designed by Eng. Meduna, was realized between 1862 and 1866 during which substantial local rebuilt was performed. This is an important

document because for the first time the conditions of the tower were described, and the decay of the wooden structure of the roof were highlighted.

For the first time ever an official document reported the presence of serious static problems in the area of contact between the bell tower and church. These problems appeared to be more evident during the execution of the works: the movement of the ashlar and the deformation of the first arch of the aisle brought, as a consequence, a crack in the masonry wall above, and this required the use of a strong support for the arch that remained in place until 1881, when the reconstruction of the cross vault close to the bell tower made it possible its removal.

Particularly the work carried out on the stone arch did not fulfill the expected goals, as the arch was propped again during the latest works in order to limit unpredictable damages on it. Its early reconstruction did not prevent it from further differential movements between its ashlar. The rash decision to demolish and reconstruct the arch during late XIX century was without a doubt a mistake and was most likely forced by the belief that there were no differential settlements between the tower and the Basilica, or that they were somehow being eliminated.

With regard to the reconstruction of the arch not much is known through written documents of what has actually been done but the direct observation of the architectural elements has provided some clarifications: the ashlar stone of the arch were for the most part dismantled and replaced with new elements in accordance with the traditional technique: almost dry joint with the interposition of a sheet of lead as interface.

In November 1864, an additional survey of restoration works stated that more interventions for the Miani's chapel were needed. This new document declared the impossibility of new foundations, as excavation at 2.40 m from the paving level revealed silty soil with no bearing capacity. Thus the need to build a robust arch, set against the foundations of the church from one side and the bell tower on the other, to unload some of the weight. Therefore, a first bond was implemented at the foundation plan between the two buildings, with consequential alteration of the structural behavior of the complex bell tower-chapel (Figure 5-1 right). These drawings distinguish the original foundation of the tower and massive brick relieving arch originally executed in 1864, which unloads the weight of the wall of St. Peter Chapel which relied on one hand on the existing foundation of the chapel itself and on the other on the foundation of the tower. Finally, in another drawing, the work performed for the foundation expansion towards the "*Campo dei Frari*" in 1903 is represented. This work has been realized with cast in place concrete connected with robust scarfs to the original foundation.

The results of this intervention, documented with a topographical survey performed with carved benchmarks on the walls of the Basilica, was of great help during the most recent phase of the works. In fact this data showed a reversal trend of the out-of-plumb of the bell tower: if before enlarging the foundations the trend was to slowly lean towards *Campo dei Frari*, with the precious help of the above mentioned survey, after the intervention, the tower leaning seemed to stop and almost reversing the movement, bringing the tower back to its vertical position. This tendency is

confirmed by the results of the precision topographical survey carried out since the beginning of the recent intervention phase. However, from the results of the latest survey, the configuration of the tower still presented a leaning towards *Campo dei Frari*, in a continuous slowly progression.

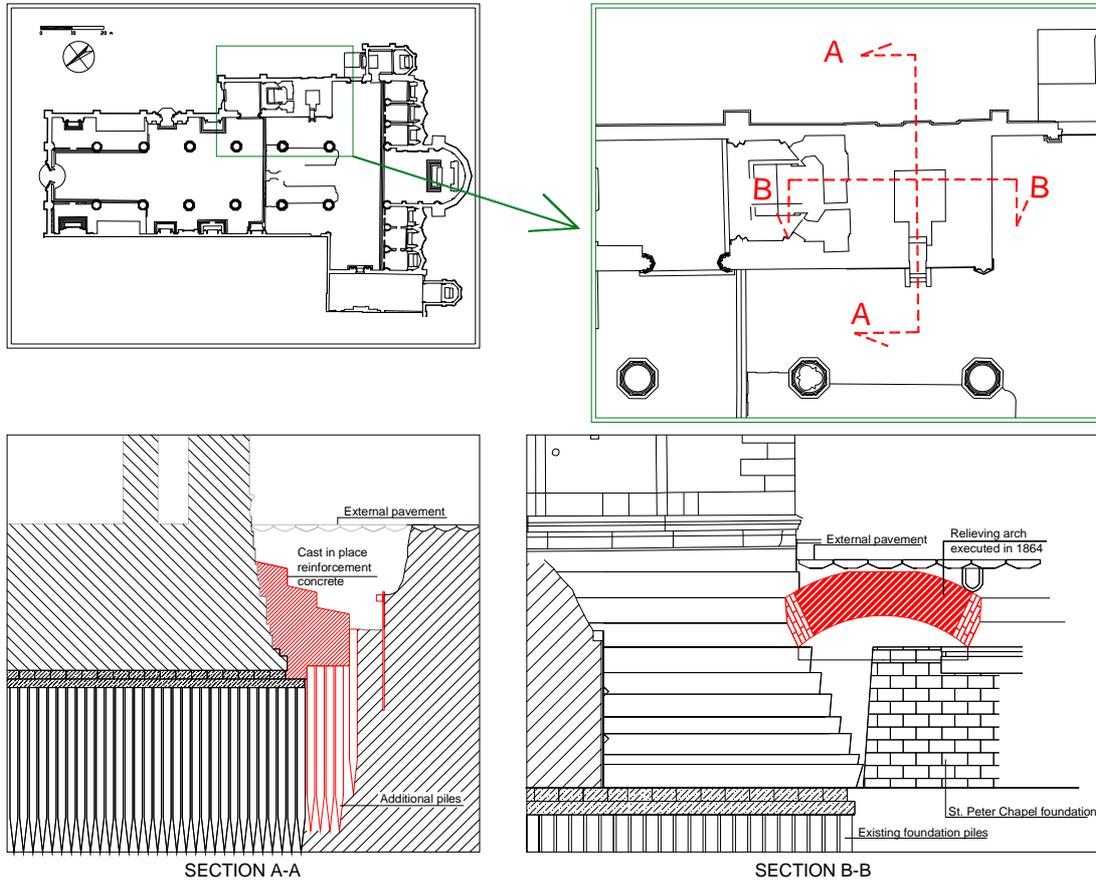


Figure 5-1: Foundation works in 19th (right) and 20th centuries (left)

More uncertain apparently was the formation of a relieving arch throughout the thickness of the wall (Figure 5-2), immediately above the arch; the internal elevation of the transept, completely without plaster, makes legible the presence of a relieving arch which certainly does not affect the entire thickness of the wall.



Figure 5-2: Relieving arch

5.2.2. XX century renovation works

The documents cited thus far from the nineteenth century, whether they are inspection reports, specifications or draft work, highlight that the bell tower and church were in need of continuous restoration but above all, especially since mid-nineteenth century, they point out the seriousness of the static problems. Nineteenth century restoration works failed to put the building in stable condition and their “repairs” to the errors were only from the visual point of view, not from a mechanical one. Only in 1902, after alarm caused by the collapse of the bell tower in St. Mark square, did the persons in charge proceed to carefully examine the static condition of the bell tower and the Basilica and set forth with a new and radical plan of renovation.

Related studies found a tendency for the church to open up in the longitudinal direction of the church, probably due to the lack of buttresses (on which you should unload the thrust of the lateral arches), and the consequently resulted in the opening of the vault arches. The report also stated that the uneven settlement of the foundations of the church and the bell tower happened not for negligent construction themselves, but rather because they had not taken into accounts the diverse distribution of the masses. The sinking of the tower was greater than 30 cm compared with the one of the church and had caused a tilt of 76 cm above a height of 42.5 meters wards the free corner of the tower.

To remedy these weaknesses, the foundations were reinforced along the outer walls of the bell tower, distributing the weight on a wider area through thicken piles of larch in order to tamp the mud and prevent the soil from continuing slipping sideways as a result of run-off of water. Then a cast-in place reinforced concrete was provided in order to make this enlargement cohesive with the original stone foundation (Figure 5-1 left). The following step was then to repair the cracks from bottom to top and from bay to bay, starting from the main façade of the church. A system of iron tie rods was then implemented between the arches of the aisles in both longitudinal and transverse direction in order to strength the entire building. This is the same procedure which originally the unknown architect had tried to do with less effective big wooden beams attached with iron anchors.

In order to preserve the authenticity of the building, the iron tie rods were hidden within new timber beams.

5.2.3. Most recent intervention

The most recent campaign began in 1990, by the Landscape and Architectural Heritage Office of Venice, and is still undergoing improvements.

During its early stages an overall understanding of the behavior of the structure has been conducted by performing flat-jack tests. This situation allowed for the designing of a campaign which was strictly related to the minimum intervention and to the maximum flexibility in order to not only correct the work during its execution but also to guarantee the gradual development of themselves. These are conditions that better ensure compatible results with the requirements of conservation of historical monuments. Soil-structure interaction is the most common dangerous mechanism that affects historical structures especially with the inadequate bearing capacity soils as found in Venice.

Combined with the direct monitoring of the behavior of the structure, soil fracturing had been carried out mainly by simultaneously working in opposite positions with respect to the bell tower, assuring a symmetrical behavior of the foundations. This however does not mean that it is not necessary to put in place precautionary provisional measures. By intervening on the foundations, the possible future configurations were taken into account to design the temporary supports. One of them is without a doubt the possibility of increasing, with an erroneous injection, the out-of-plumb toward *Campo dei Frari*, resulting in finding a configuration similar to the one that was occurring before 1902. This situation would have led to an increased crack pattern linked to the separation of the two structures and consequently would worsen the stone arch that separate the transept from the left aisle. Besides, a favorable decreasing of vertical load and flexural moment affecting the column would occur. This was the main objective of the final intervention and the procedures to pursue this task will be further discussed.

To avoid the occurrence of these two dangerous situations, three main devices were put in place. First: the already troubled stone arch was propped with a double timber truss. The same configuration was already used until 1881 and it seemed to be the best solution to adopt in order to avoid partial collapse of stone ashlars. The prop structure spans from the NW corner pillar of the bell tower to the last column of the main nave (Figure 5-3).

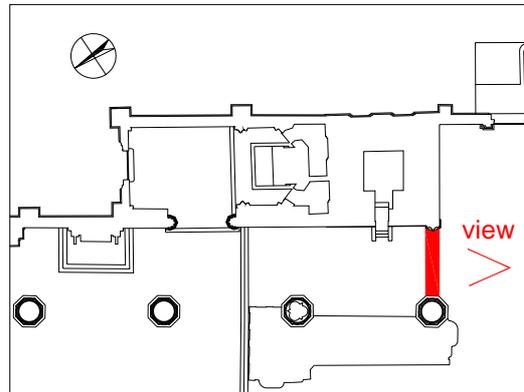


Figure 5-3: Propped stone arch: position and picture

Second: two more \varnothing 58 mm tie rods, totally compatible with the pre-existing ones placed at the beginning of the XX century, were located above them in order to maximize the tying effect. Taking advantage of the room created between the twin tubes of the structure, a hole was drilled on NW and SE walls where the ramps are placed. To facilitate the in situ works, tie rods were divided in two pieces, connected with a turnbuckle placed inside the tower (Figure 5-4). Particular care was placed in manufacturing the metal tie plates in the same shape of the pre-existing ones.



Figure 5-4: Tie rods

Third: to thwart and absorb the tower mass that would have leant towards *Campo dei Frari* a temporary steel tie rod was therefore designed. This proposal had the intent of minimal intervention on the structure by installing a device that would be totally removable without instauration of damages in the structure and with the goal of preventing further differential movements between the two structures. The \varnothing 24 mm steel cable was put in tension for this purpose and its location

was chosen in order to maximize the stabilizing effect on the column, which, as explained earlier, is a strong inflection towards the interior of the Basilica (Figure 5-5).

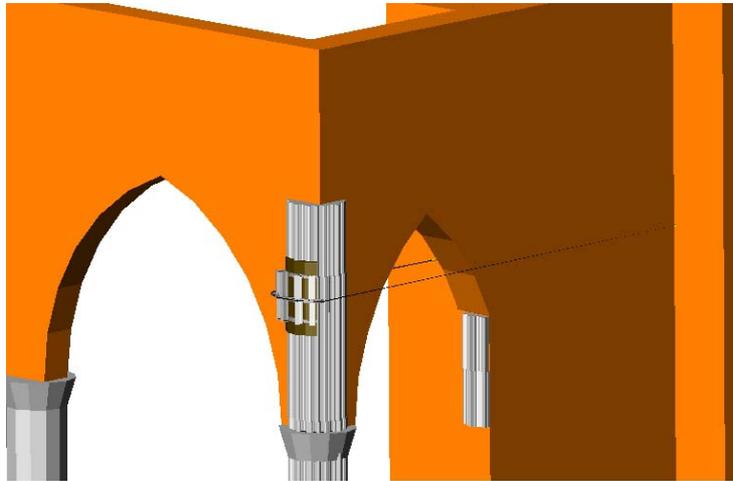


Figure 5-5: Provisional steel cable

The metallic steel cable was housed in a curved steel beam that surrounds the column. Not to create local damage on the pillar by punching loads, a soft wooden protection has been placed between the metal structure and the stone column. Moreover, to uniform the distribution load, four HEA100 steel beams were placed vertically between the cable and the wooden protection (Figure 5-6).

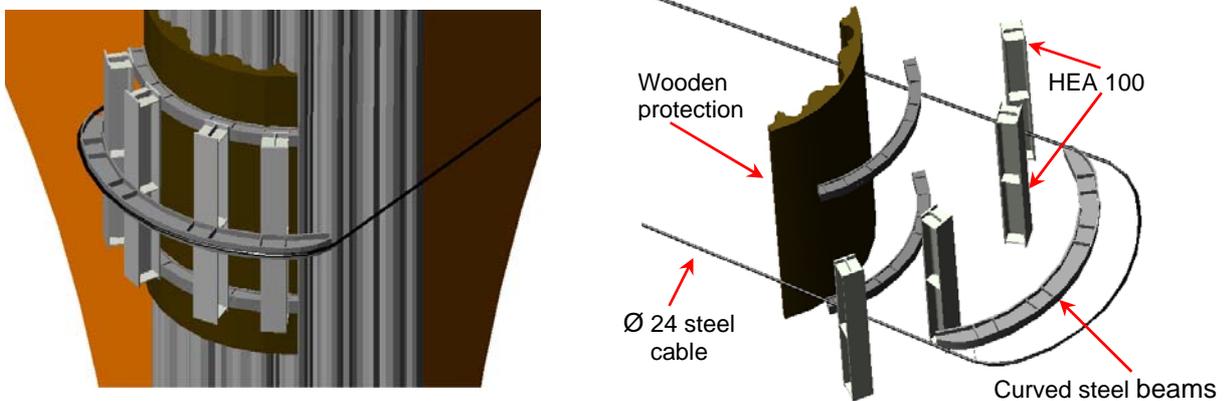


Figure 5-6: Exploded drawing of the device at the column

On the other side of the cable, on the SE wall of the tower, the removable tie plate was made by two UPN profiles stiffened with 10 mm gusset plates. As well the masonry wall was protected from local damages by a 2 cm thick steel plate that enlarged the load transmission area in order to stabilize the stress value around 1 MPa (Figure 5-7 and Figure 5-8).



Figure 5-7: Provisional tie plate on the tower

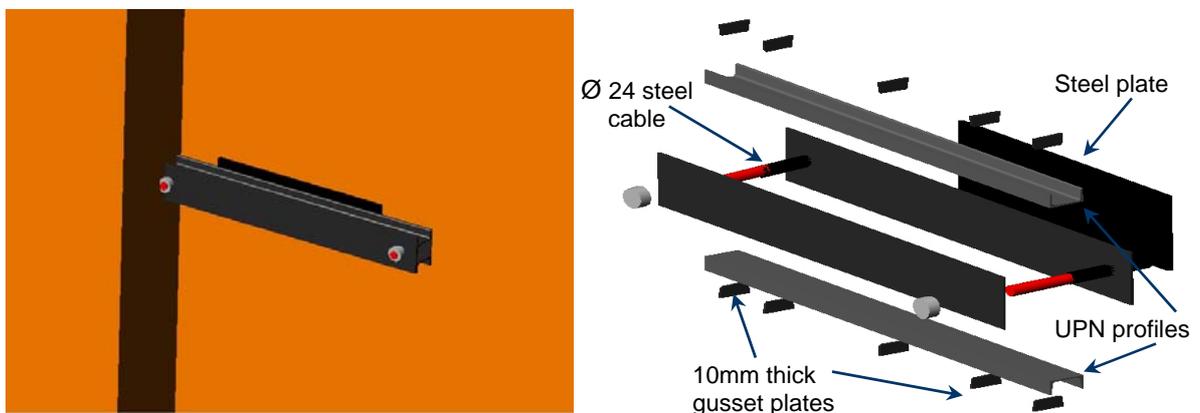


Figure 5-8: Exploded drawing of the tie plate at the tower wall

Towards the center of the metal tie rod, inside the bell tower, a steel turnbuckle was placed, in order not to put a pretensional state on the cable but to accommodate three strain gauges to measure the deformation of the steel rod clearly related, through the elastic modulus, to the load which affect the cable. The purposes of this steel cable is hence clear: the vertical component generated by the tower rotational movements is still absorbed by the column, while the horizontal component is transferred to the steel cable which unloads the force on the steel tie plate outside of the tower.

The position of the steel cable was such that it minimized the existing flexural moment acting on the column and maximized its effectiveness by increasing the distance from the base of the column. The position is therefore at about 14.4 m high from the ground. This latter intervention is certainly one of the lightest and also offers the advantage of contributing monitoring the structure: measuring the strain variations in the tie rod is a clear indicator of the onset loads that the two adjoining structures exchange between themselves and a sign of possible undermined stability of the superstructures.

As mentioned before, these three were only provisional strengthening techniques; the main problem of the settling structure was handling by the soil fracturing intervention.

The first phase of the work aimed eliminating, or at least stabilizing, the causes of the settlement phenomenon. The results from the monitoring system show that this objective seemed to be fulfilled, but it may have been too short of a data acquisition period to firmly state that the problems of the tower were over, even if the trend undertaken by the measures seemed to lead to this conclusion.

This first stage of interventions on the Frari's Basilica, from the beginning of acquiring information to the end of the injections, lasted almost seventeen years and saw the collaboration of great experts in a wide range of engineering fields.

A second stage of the works, which latest favorable results encouraged the experts to undertake, started in May 2008. In this period a meeting took place between the representative of the Landscape and Architectural Heritage Office of Venice, structural and geotechnical engineers and the construction company which is following the works on site. During the meeting specialists agreed on the chance to physically separate the structure of the bell tower from the main body of the Basilica. As far as the documents achieved concern in fact, bell tower and Basilica where built separately: following connections were performed only during subsequent phases. And it is this very reason that drove the experts to the following stage.

To physically separate the two structures has quite an important effort and to be sure of the validity the documents cited, evidence had to be detected. Clues on where to find a separation structural joint were discovered just behind the transept wall above the last cross vault on the left aisle.

By removing the first two courses in a sample area of the inner corner side of the transept wall, evidence showed that the bell tower was only superficially connected; after removing a sample in the red highlighted zone of Figure 5-9 the proof was given.



Figure 5-9: Sample zone

After the execution of a sample hole, performing the cut between the two structures implied an extra effort. In fact it is of extreme importance to monitor the work during its execution. Not only great care must be put in dividing the two structures, but also the right amount and position of new

monitoring spots must be provided. The precision topographical survey carried out day to day will provide excellent information, but also already active crack meters, pendulum and other gauges will also grant reliable information. For example it could be necessary to perform additional flat jack tests to validate the expected reduction of uneven compressive stress in the notorious column. In fact, by separating the two structures, vertical load on the column should decrease by a significant amount, as would the horizontal load which therefore would decrease the flexural moment. Only after a long term monitoring period, during which the stabilization of the movements is confirmed and the cracks are inactive, will it be possible to establish a final project. As well a corresponding definition of interventions on the super-structures will be available, and, as already mentioned, improvements performed based mainly on traditional techniques, more environmentally friendly for the historical construction.

5.3. Soil fracturing

Moving again the attention towards the soil-fracturing technique, which aim is to solve the problem at its root, it can be stated that is based on the behavior of the tower after the work of 1902, when a limited foundation enlargement (either by extension, having only interested the SE side of the tower, and by effect, considering the weak scarf with the existing structure) highly affected the rotational movements of the tower.

The proposed solution to solve the problem from its root is the *soil microfracturing* performed around the bell tower. This technique is performed in order to obtain improvements in stiffness and strength in the soil by tamping its bulb (where insufficient wooden piles act) which was the main cause of settlements.

This technique has the great advantage of allowing the injections to be performed as gradually as necessary depending on the particular situation. In fact on the basis of a careful control, carried out continuously during all stages of processing, the soil-structure interaction, assigned to an adequate monitoring system, is continuously checked. The continuous monitoring action is important not only to check the quantity and the modality of the injections, but also to immediately stop the works if any unpredictable phenomenon would have occurred. This intervention, considered the most suitable for the work and the least invasive, is extremely flexible (Figure 5-10).

Moreover, the continue monitoring provide essential guidance for the calibration of the parameters and injection of the ongoing works.

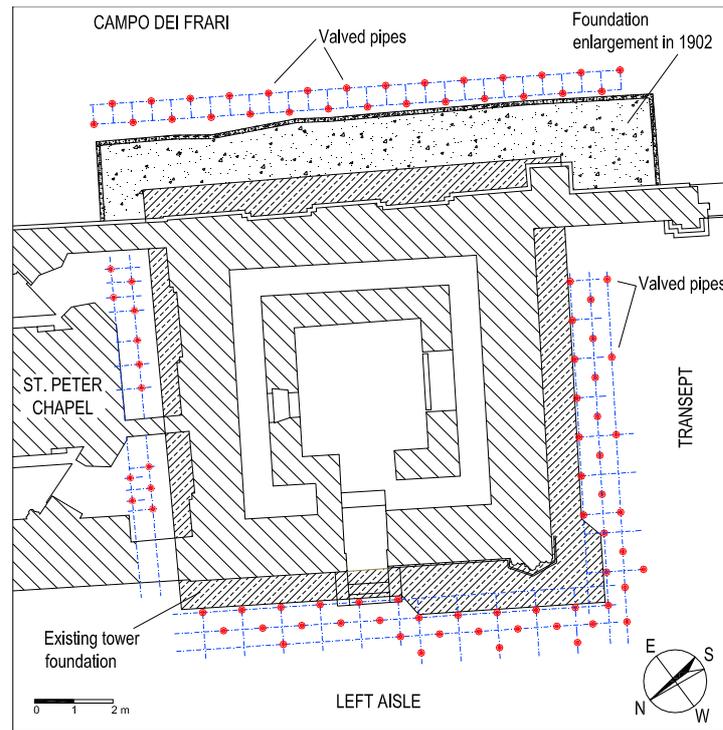


Figure 5-10: Soil-fracturing scheme

The execution of soil fracturing allows the selection of different operational options: the work plan can foresee the execution of injections on individual pipes, groups of pipes and most of all it allows to choose the depth of the intervention. Moreover this multi-option work allows stopping the injections in a particular pipe and continuing it couple of days later, during a different stage.

The goal of the soil fracturing was to obtain a confinement of the soil. In particular, improving the deformation modulus of the soil, with consequent reduction of deformations deferred over time (creep), and increasing the mechanical properties of shear resistance were the principal objectives. The intervention of consolidation of the foundation soil of the bell tower of Frari's complex, carried out between March 2005 and May 2006, consisted of installing 88 valved pipes. These piles were placed all around the bell tower in staggered lines in order to maximize the area of the intervention. Along half perimeter of the bell tower three lines of valved piles were installed while on the SE side, towards *Campo dei Frari*, where boreholes discovered the enlargement of the foundation (executed in 1902), only two pipe lines were used, as close as possible to the existing foundation of the structure. Due to space limitations on NE wall, towards St. Peter chapel, the intervention provided only one line of valved pipes, with the addition of some pipes on a second outer line only towards the end of this line up (Figure 5-10). The valved pipes are 12 m long, 12.5 mm thick and with an outside diameter of 88.9 mm; each pipe has 21 valves, placed along the pipe every 50 cm in order to better control the intervention and direct all the efforts in strengthening as much soil as possible. Work proceeded slowly so as not to create unexpected situations and to induce the least disturbance possible to the soil.

A picture representing a portion of the piles is depicted in Figure 5-11.



Figure 5-11: Valved piles of the soil-microfracturing technique

However, not all the layers of the soil were treated in the same manner. Quantity and typology of injected mixture and pressure were different among sandy and clay layers in order to favor a targeted intervention for each substratum.

For the injections, a particular mixture was used. In particular it was composed by pozzolanic cement, bentonite, calcareous inert and water (plus additives), injected with controlled pressure, each one regarded the typology of soil in which it had to be injected. This mixture was selected among different compositions that were tested during early stages of the work. In fact, before selecting the soil-fracturing technique, local tests performed on the soil were giving accurate response on the reliability of the method.

The settlement monitoring during the intervention was an essential part of the methodology of intervention: in fact there is no way to implement conservative measures to safety conserve the huge mass of the bell tower and the enclosed Basilica from an unpredictable behavior of the structure.

However, if the stabilized trend of the latest results is not confirmed for a long monitoring period it could be necessary to draw on the extreme resource of deep consolidation which is more expensive. This intense technique would take utilize injecting pipes up to the deepest layer of the soil thus transferring significant portions of vertical load to the deep sandy soil.

More information about this topic can be found on specific technical reports draw up by the firm who carried out the works and which references are reported at the end of the present work.

6. FINITE ELEMENT MODEL

The best model of an historical monument is the historical building itself. Load history distribution, crack pattern and involved peculiar phenomenon of a monumental construction makes it unique among other edifices of the sort. However, a finite element model is important in order to better estimate the behaviors and trends of the structure. Due to the high complexity of such historical structures, especially caused by the difficulty of modeling non-linearities of masonry properties (caused by their infinite inhomogeneity), lead the model to get close to the reality but not quite equal it.

However, a computational model can be necessary to evaluate past structure behaviors, such as those due to different load distributions or behaviors during its construction phases. Moreover, an accurate finite element model can serve to represent not only important aspects related to the structure but it can also predict future behaviors, for example, with the addition of the proper modeled strengthening techniques. Computer models can also be very important in assessing possible scenarios especially related to the phases in-between the works of consolidation, in terms of appropriate stresses or settlement values to ensure adequate safety margins during all the operational processes.

Finite element models are without a doubt important tools in acquiring information. They may be helpful in locating tensional states that lead to or may have resulted in the formation of a crack which the damage survey could have been missed or they can give evidence of an unknown ongoing mechanism. As for the high inhomogeneity of the structures, in order to be reliable, models must be carefully *calibrated*. Calibration implies the knowledge of the mechanical properties of the materials (such as density of the material γ , Young modulus E , Poisson coefficient ν), or other parameters such as frequencies or modal shapes, each depending on what are the purposes the operator wants to fulfill with the model. An imprecise calibration of a FEM can mislead the operator and therefore result in improper conclusions.

Bearing this in mind, some finite element models were developed for the Basilica of Frari. Although some limitations arising from the practical impossibility of having data describing all the mechanical properties of a big complex as the Basilica's structure, those FEM proved to be powerful tools for structural safety assessments.

Unfortunately, due to lack of time, further developments of the existing models could not have been performed in this thesis; as known, modeling is a costly (in time) procedure and it also requires suitable working machines.

Therefore in this chapter some of the latest results coming from the previous models will be briefly discussed (note that the reader can find the complete literature on this topic on external reports). Moreover, possible further development will be discussed in order to underline future extension of the work, while taking into consideration the latest phase of the ongoing works on the Basilica.

6.1. Description of the model

A finite element model utilizing the software Straus7® was built by Eng. Filippo Casarin and Professor Claudio Modena final year student during 2003. This model concentrates its efforts on describing the part of the structure affected by the soil settlements. Figure 6-1 represents the bell tower, the adjoining walls (where benchmarks and related data are available) and the left aisle of the Basilica from the 3rd by up to the transept including pertaining vaults, arches and tie rods.

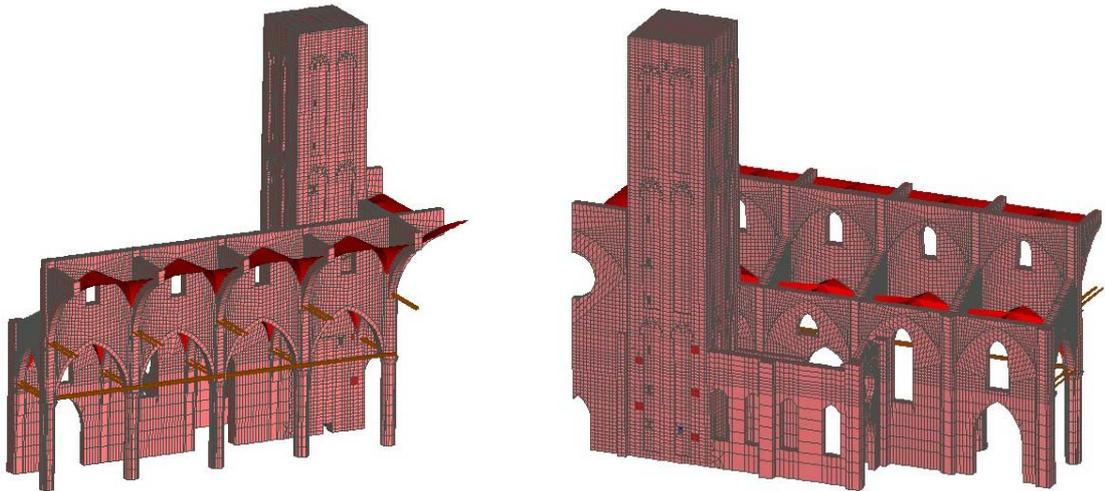


Figure 6-1: Latest FE Model

It is important to mention that in order to not model the entire structure, some parts of it have been described as punctual or uniform loads. This is the case, for example, of the belfry and the wooden roof. These portions have been substituted by loads in order to save computational time without actually losing information that would have affected the results on the Tower and Basilica's output (Figure 6-2).

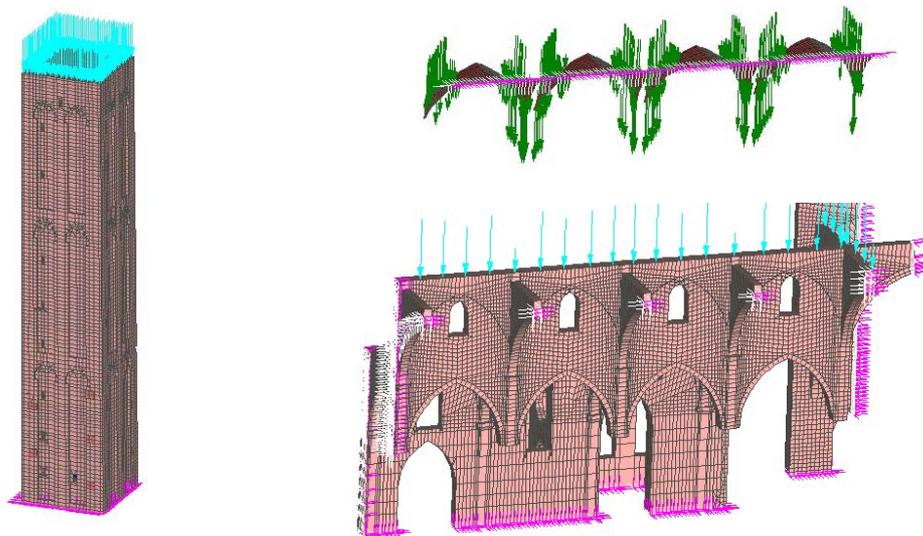


Figure 6-2: Loads representing not modeled structures

Symmetrical restraints were used to avoid modeling the mirror part of the church thus avoiding longitudinal and transversal differential movements and rotations. This procedure, commonly used in symmetrical objects, allowed to better refine the mesh in the most critical parts of the building in order to obtain more accurate results.

In these models, differential settlements and related values are taken as input data during the analysis; therefore, analyses are performed with the goal of assessing the effects on masonry structures of the differential settlements of the soil-structure interaction.

The objective of finding stress values and close to true values of settlements and displacements led to the consideration of the option of a sort of step by step analysis. By using documents and data achieved from the beginning of the works, four main phases can be distinguished:

- an initial phase reflecting the completion of the complex, approximately in 1450, in which considering the undeformed shape of the Basilica even though settlements could have been already occurred;
- phase referring to the condition of the structure until 1902 when the enlargement of the foundation was performed;
- condition of the complex up to 1990, when the first modern intervention phase took place;
- current situation including existing crack pattern, up to 2000.

Pursuing the final configuration of the computational model was achieved in incremental steps, each started from the results of the previous one by updating settlement data (imposed movements to the joints) and/or revising the model including new structural features (such as modified properties or new beam elements to represent new tie rods).

Hereafter, some of the main conclusions and results of the finite element model are reported.

Figure 6-3 shows the stress patterns in the principal directions. The transversal direction considers the behavior of arches, vaults, walls and buttress warps in X direction and lying on the XZ plan of the model. In this direction we can see that all parts of the walls have a uniform distribution of tension. Different behavior can be noticed however in the transversal buttresses of the main nave comparing them with the same structures of the left aisle. This behavior can be traced back to the heavy thrust of the main nave arches. These thrusts are probably not properly referable to the tie rods; hence the partial flying arches give their contribution to the safety of the structure by transmitting the loads to the buttresses of the lateral walls.

Longitudinal behavior (Y direction) is not of particular interest for the sake of the arches which lie on the YZ plan: the horizontal thrust among a series of contiguous arches usually balances itself. Only the last arch on the 3rd bay shows an uneven behavior but this is due to the presence of the nearby restraints. Otherwise noteworthy is the behavior inspected from the longitudinal differential displacement along Y direction (Figure 6-4). The largest displacement, towards the transept, is surely the one in the upper stem of the last column of the Basilica. The thrust of the arch above this column is not counter act by another arch, and the tie rod placed between the two capitals seems not to be working properly.

Finally, regarding the vertical direction (Z direction), it can easily be noticed that most parts of the structure are subjected to compression. As predicted, the most stressed parts are the slender ones (columns) while the others, due to their considerable mass (tower and walls), show a lesser stress value. The columns, especially the one towards the transept, show an uneven compression state. The reason for that has already been explained in previous chapters.

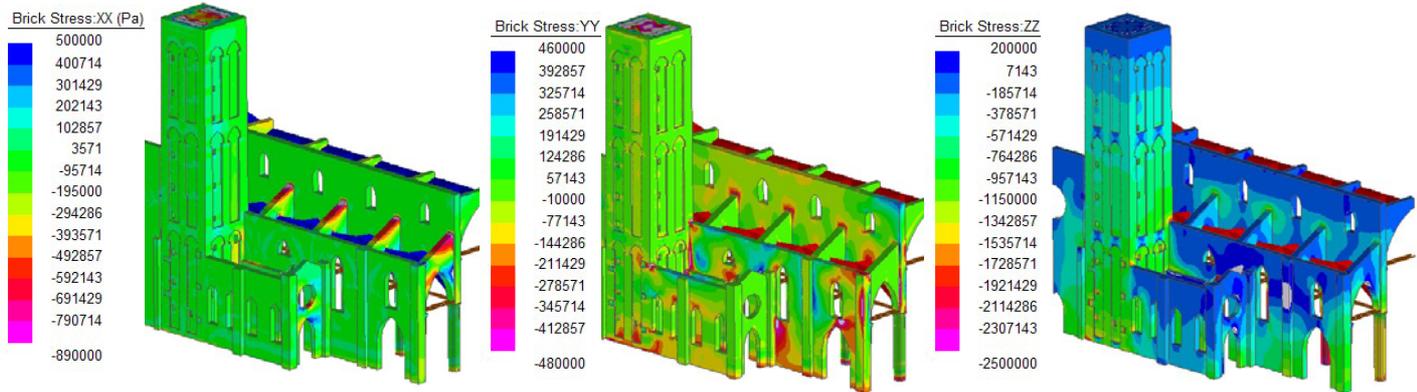


Figure 6-3: Stress in XX, YY and ZZ direction

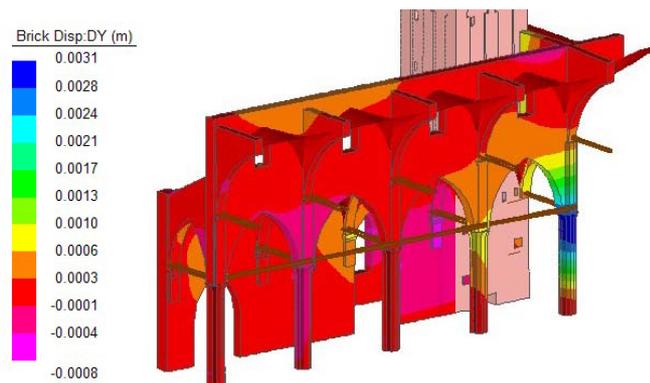


Figure 6-4: Displacement in Y direction

These results accurately show the actual trend and state of the Basilica together with the bell tower. A finite element model of such an elaborate structure could therefore be reliable only qualitatively, but not always quantitatively due to the numerous variables that play a key role on the real structure such as local phenomenon, which are very difficult to model.

Taking into account the dimensions of the Basilica, in order to save time and money while at the same time not lose important information on the behavior of the structure, the decision of the operators was to conduct a linear analysis considering the masonry material as homogeneous, isotropic and linear. FEM analysis allows for the determination of the tensional state in the structure, but in most cases do not detect a reasonable behavior of elements subject to tensile stress. FEM does give the overall building stress distribution, but does not give hints on the structural safety of the construction whenever no tensile bearing load materials are present.

Non-linear analysis is a time and cost operation and leads to further problems whenever *non convergence* of the analysis happens. “Non convergence” would mean that material exceeds its tensile strength in localized parts of the structure and computational efforts fail in completing the operation. This implies revising input data of the model in order to achieve final convergence of the procedure by, for instance, changing the iteration method or refining the analysis step.

Mechanical properties assigned to the materials of the model have been achieved mainly from the flat-jack tests results (double flat jacks provided the modulus of Elasticity in different points of the tower and transept wall), but this was not enough. In fact these values needed to be calibrated (thus the model) in order to obtain values as precise as possible. As mentioned above there are usually different ways to calibrate the models: in this case, starting from the values attained with the flat-jack tests, the Modulus of Young of the masonry was varied to reproduce numerical stress values close to the reality and a crack pattern close to the one deducible from the damage survey.

Non-linearities were introduced in the model following an iterative process that roughly simulated the crack pattern in a linear fashion: by carefully reading the results of the first model (first step performed by introducing settlement data), the material of the elements in which tensile stresses reached a common limit value for masonry structure was replaced with a “softer” one, with a reduced value of modulus of elasticity.

Lower values of Young's modulus indicate that those elements will be less stiff in the next analysis' step. This is done in order to redirect stresses towards other parts of the structure, knowing that those modified elements cannot contribute anymore to the normal behavior of the building (Figure 6-5).

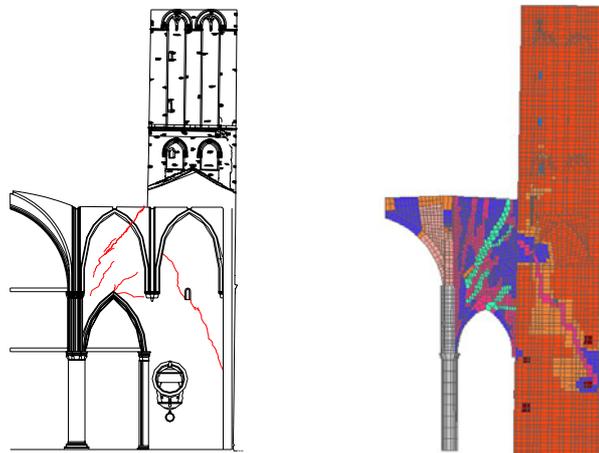


Figure 6-5: Crack pattern (left) and different Young's modulus in the model (right)

The following step model, modified in its physical properties, is then subject to the updated settlement data and the analysis follows this procedure until necessary, in other words until reaching the actual configuration.

In Figure 6-6 iterative property changes in the four created models are depicted. These models show the walls of the transept and the left aisle, both seen from North direction. Cyan diagonal

elements depict the well known crack found from the damage survey in correspondence with the relieving arch above the masonry vault at the end of the left aisle. Pink brick elements are the ones that do not suffer damages and for which properties are defined since the first calibration procedures. After the first step analysis, with the support of the historical documents possessed by the Landscape and Architectural Heritage Office of Venice, accurate redefinition of the properties could have been performed.

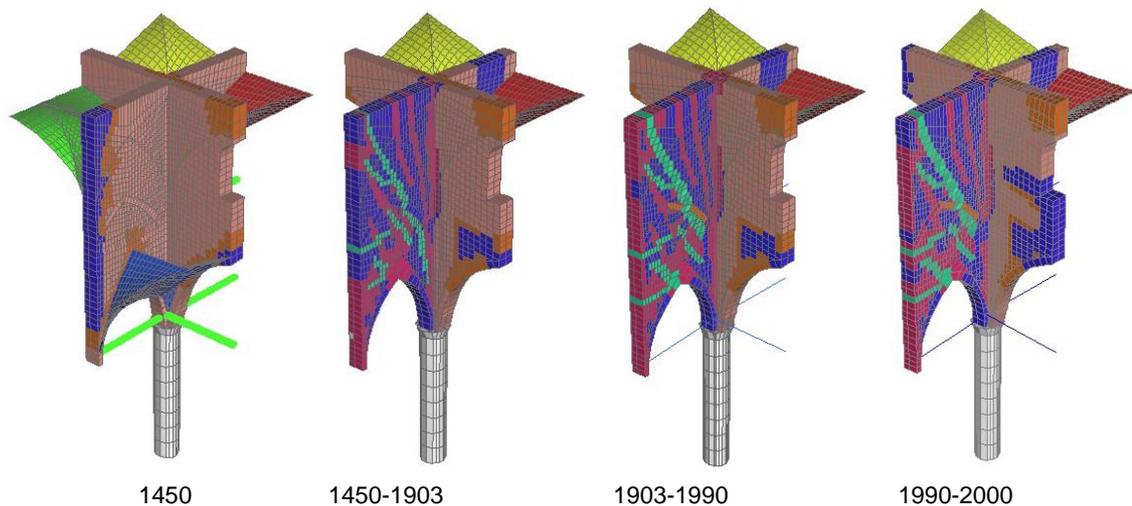


Figure 6-6: Different properties among the finite element models

Moreover, in the same Figure 6-6, it can be noticed that two vaults only appear in the first finite element model, while they have been removed in the latter. The main reason for this operation is certainly related to the fact that vaults do not show a completely clear behavior in a linear elastic analysis and they suffered high tensile state, obviously not tolerable by masonry vaults, which influenced the results of the stress pattern on the nearby walls as well.

Furthermore it can be noticed that the different configurations of the tie rods, which initially were timber made (first model), were later substituted with metal ones (third model).

Massive historical monuments were mostly built using an empirical method, relying on information handed down by Master masons, generation to generation. Materials do not work at their maximum strength capacity as these massive buildings (especially churches) usually have a very high safety factor. Therefore stresses are generally low. It may occur that only some localized parts of the structure show high stress concentrations causing confined damaged that however do not induce global collapse mechanisms.

All these mechanisms could have a better interpretation with a non-linear analysis, which implies the definition of an accurate constitutive law for the materials: stresses and strains can predict accurate crack patterns where the tensile strength is exceeded.

In the model of the Frari's Basilica however, the same concepts were applied to the structure by using only linear laws for the materials involved. This means that no tensile limits were set and the

model itself did not immediately show clear results of a possible ongoing crack pattern. The process to determine it was necessarily a step by step iteration procedure with continuous feed back with historical information and past surveys concerning damages to the Basilica. It was not easy to model the structure as it was immediately after its final construction and no documents clearly stated the damages during that period.

From this perspective, the four phases of the modeling process can now be explained, leaving to the reader further reference information for detailed results.

6.2. First results

As a first approach to the structure, only the closest part leaning to the bell tower has been modeled, including the last column of the nave, the portion of the transept from the column to the tower and the 6th bay of the left aisle (Figure 6-7). This part was chosen because it is the most directly affected by the movements of the tower, as many restoration interventions during the centuries have shown.

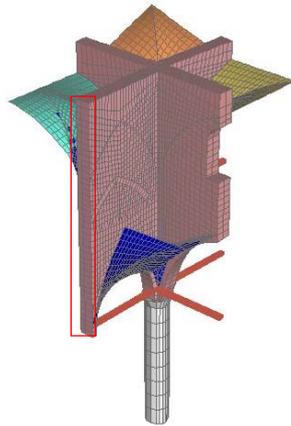


Figure 6-7: First sub model to be analyzed

Following the phases depicted in Figure 6-6, the first model to be examined is the one which attempts to represent the existing conditions around the mid-fifteenth century. In order to avoid wrong distribution of loads between the bell tower and the most stressed column, a linear displacement law was assigned to the joints at the interface of the elements of the first bay of the left aisle (yellow highlighted in Figure 6-7). Previous analysis demonstrated that a fix restraint for these nodes would have meant a high stress concentration towards the column, which would have been unrealistic. Masonry vaults were modeled with a thickness of 30 cm, density of 2000 kg/m³ and Young modulus of 3.3×10^9 (Figure 6-8a).

The second phase was modified in order to represent the maximum out-of-plumb of the bell tower approximately in 1902. Not having implemented the structure of the bell-tower (only in a following model), in the same manner as before, interface nodes were implemented with two linear displacement law, one in Z and one in Y direction, to schematize the effect of the out-of-plumb.

These laws have been studied with the help of the data of the documents of that period (Figure 6-8b).

The same procedure was used for the third model in which the configuration after the enlargement foundation work was represented. Extremely important was the comparison of the topographical survey of 1902 (with carved benchmarks placed all around the church) and later topographical survey achieved at the beginning of the modern works. Further improvement achieved in this model was the substitution of the timber tie rods with 45 mm diameter metallic tie rods (Figure 6-8c).

The fourth and last model wanted to represent the situation right before the soil intervention. There were no changes in material properties because of the limited window of time that would have liked to be represented in this model (only ten years, from 1990 to 2000). Results were slightly different from the previous ones as the settlements over ten years were not sufficient enough in order to create a totally different tensional state.

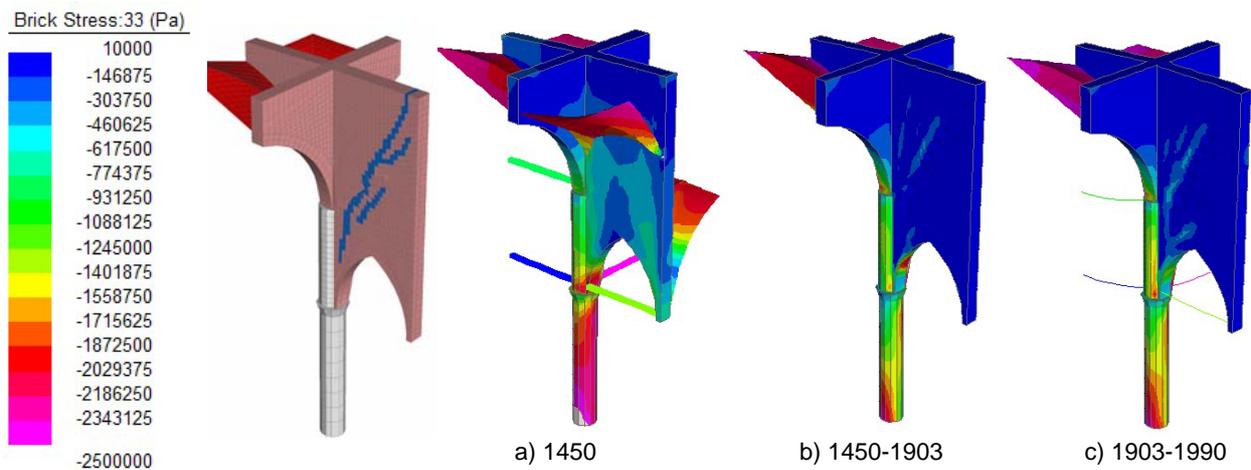


Figure 6-8: Principal minimum stress (3-3, compression)

Further improvement of the finite element model was then to implement the whole bell tower which would have replaced the displacement laws of the border joints. Figure 6-9 depicts the most representative images of the model, again with different properties of the materials in order to physically represent the crack pattern.

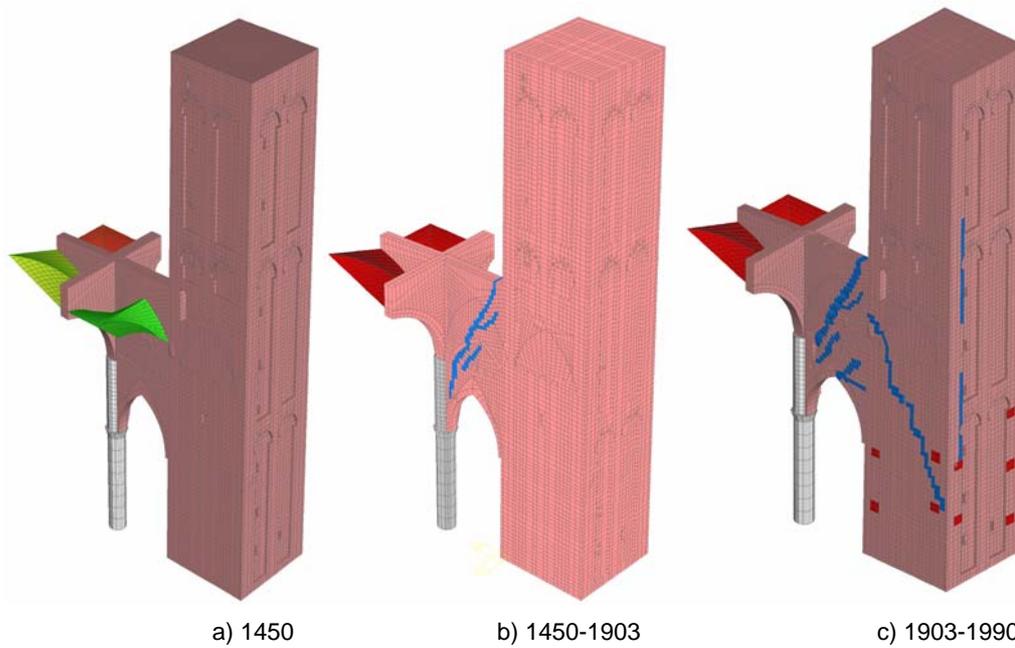


Figure 6-9: Further improvements of the finite element model

Figure 6-10 represents the principal compression stress for the most representative phases above mentioned, clearly showing the wide diagonal crack across the transept wall.

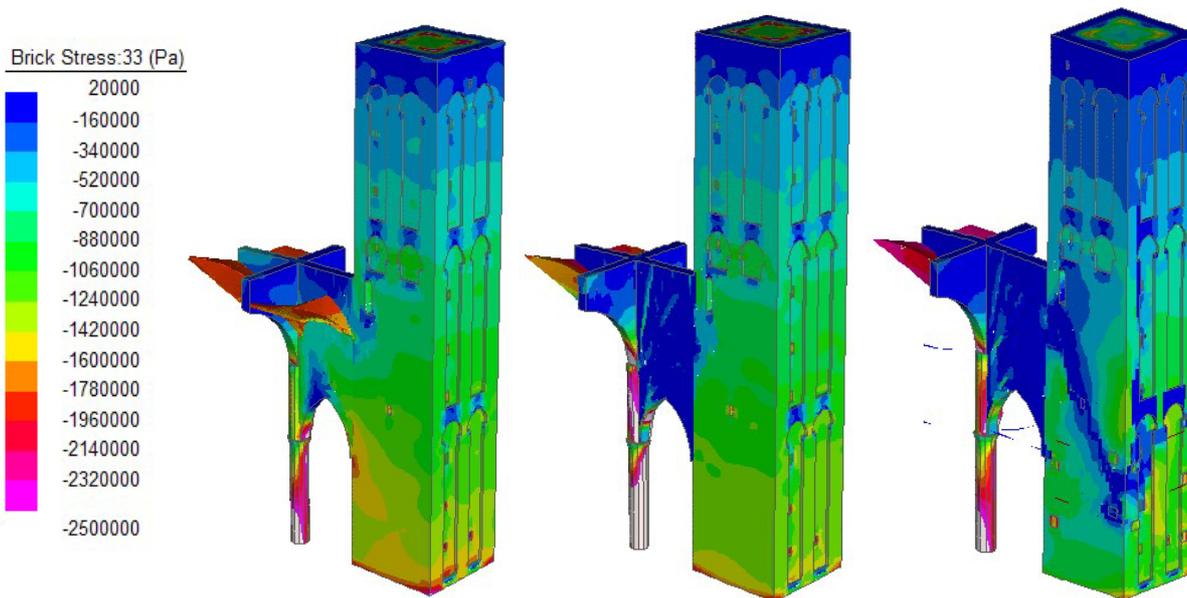


Figure 6-10: Principal minimum stress (3-3, compression)

Further improvement of the existing model with Straus7® software could be the one depicted in Figure 6-11, but unfortunately, due to the complexity and dimension of the Basilica, the mesh aspect of the elements has to be strongly reduced in order to proceed with the analysis. In fact, any attempt to run the software with the same mesh used in the previous models will fail due to

insufficient memory to allocate to the project. Refining the mesh will therefore mean losing information, most at the sake of accuracy.

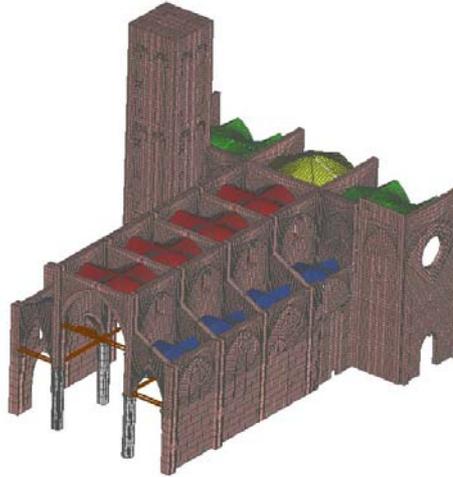


Figure 6-11: Development of the Basilica's model

A further step in the modeling features would be to perform the computational model with a powerful software that supports a reliable non-linear component. A powerful tool for non-linear analysis is the *Diana*® software, which proved to be a very high reliability software for modeling masonry structures. A more accurate analysis than the one described before, it can be carried out by performing a particular non-linear analysis joined together with a key feature called the *phase analysis*. Diana's phase analysis allows for the implementation of combining all past and present strengthening devices into one model. Afterwards each single apparatus can be activated or deactivated during each phase. For example in the same FEM both timber and metallic tie rods can be implemented. One group of beam elements will be active from the beginning (timber ties), while the second (metallic tie rods) will overwrite the previous during a following phase: properties of the beam would change automatically between the subsequent phases.

This is a powerful technique, therefore quite demanding in time execution. Attention must be paid while writing the code of the programme as each step must be strictly connected to the following one: an error in activating one system instead of another could be difficult to identify.

After modeling the structure, some examples of commands that could be assigned by conducting a phase analysis with Diana are the following:

- substituting the timber tie rods with the metallic ones;
- activate the enlargement of the foundation which was implemented in 1902;
- activate steel tie rods installed within the tower;
- activate the provisional strengthening devices.

Another approach would be to model the soil beneath the base nodes of the structure. Some of them, like the ones away from the tower, would be fixed support while others would be subject to differential settlement histories, according to both past and recent topographical surveys.

This procedure would force the model to represent particular configuration during its history; while reproducing the base settlements, hence the out-of-plumb, the overall tower/Basilica stress state could be read.

Moreover, having performed borehole soil samples, a more accurate approach instead of assigning differential displacement at the base joints would be to directly connect each group of nodes with springs representing the particular characteristics of the soil beneath (namely the stiffness). This method should lead to the same results as the previous one but gives the opportunity to further modify, again with a phase analysis, the properties of the soil by defining the data evaluated from the monitoring system installed during the soil-fracturing intervention works. The settlement should stop on the tower and the adjoining structures as the springs become stiffer.

CONCLUSIONS

- The documents acquired during the historical survey were of considerable importance in assessing the troublesome developmental stages of the Basilica. Not only had they allowed for the identification of the various phases of the construction but they also revealed the structural weaknesses caused by them.
- Despite its importance, visual inspection is sometimes insufficient in acquiring important information about the materials and about invisible aspects within the structures. Hence complementary diagnostic methods are needed, based on non-destructive or minor destructive tests in order to allow for a wide range view of the problem. From this point of view the proposed campaigns were studied in order to directly obtain the desired important results in order to justify the trend behavior of both the Basilica and Bell Tower structures. Results were decisive in order to design the strengthening technique projects.
- NDT, but most of all MDT, definitely helped in the assessment the structure. They provided information on geometry and structural anomalies and gave an overview on the actual condition of the building. The main objective of these tests was to increase the integrity of the building and let the experts learn important information (parameters and properties) from them, the most important of which is the creation and the calibration of a finite element model. Of course due to the inhomogeneity of the historical buildings and the assumptions implied these test have their limitations. As well the correlation with mechanical properties is limited but the combination of more test results on the same parameters can be very reliable.
- A key role for the interventions on the *S. Maria Gloriosa dei Frari* Basilica is played by the monitoring phase. The monitoring system has been and will be the “doctor” of the ecclesiastical complex and will continuously give clues about the ongoing phenomenon.
- Hence it can be said that a vigorous monitoring of the structure before, during and after the strengthening will give a clear picture on the structure movements and its condition. If actual soil intervention is unsuccessful, this monitoring will help in assessing a further strengthening design decision which will imply minimal intervention. The less interference the better for the sake of any successful heritage strengthening project.
- Soil microfracturing reveals itself to be the most efficient technique to be used in this historical case study among three other commonly used techniques, namely, foundation enlargement, grouting and micro piling. Combinations of some of these techniques, performed in past interventions, did not succeed in solving the leaning problem of the Bell Tower, while soil-fracturing seems to have reached this goal.
- Furthermore, F.E.M models were used to evaluate the stress configuration of the ecclesiastic complex. Due to lack of time, these analyses were not further developed and therefore remain incomplete. However the limited results obtained showed a clear picture of the ongoing phenomenon. More complete and realistic analysis (non-linear) should be

carried out and a complete model of the Basilica should also be carried out.

- Only after the main works are completed (namely soil fracturing in the foundation soil) can a complete structural analysis be performed. This final step would contemplate all the final data coming from the monitoring system and the soil may be implemented in a finite element model as well. This would lead to a more accurate result and should testify to the stabilizing of the settlement phenomenon.
- Regarding the method to be adopted for repairing the cracks, a continuous monitoring is a must as mentioned earlier in the report. After a vigorous monitoring if the structure is declared by team of experts to be safe and the settlement to have stabilized, it is proposed that the cracks be examined again. After re-examination, it is proposed only to re-point or fill the cracks with suitable mortar of suitable strength, so that the cracks do not lead to unaesthetic problems which would lead to further structural damage.

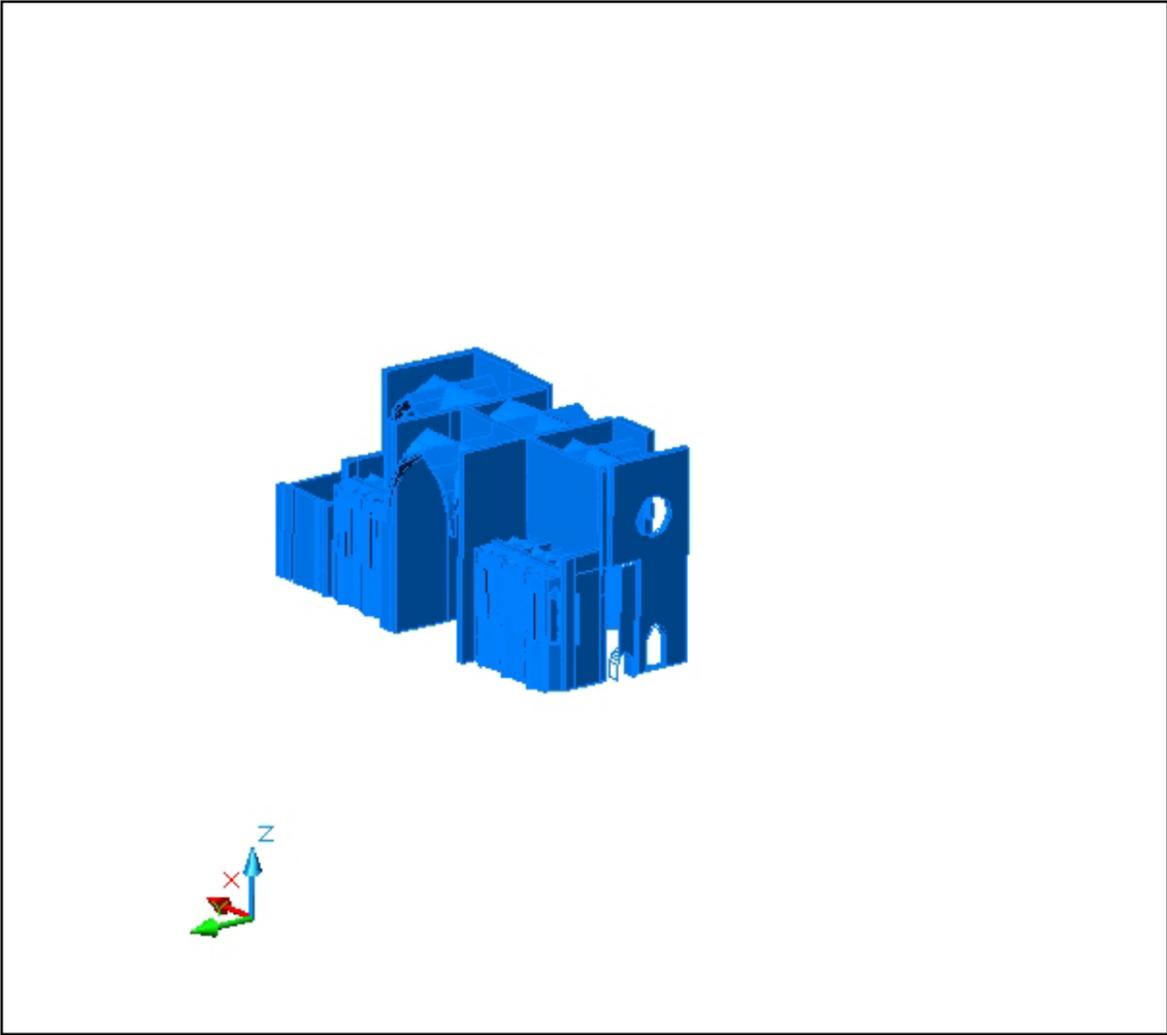
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- www.soprintendenza.venezia.beniculturali.it
- www.chorusvenezia.org
- www.archivi.beniculturali.it

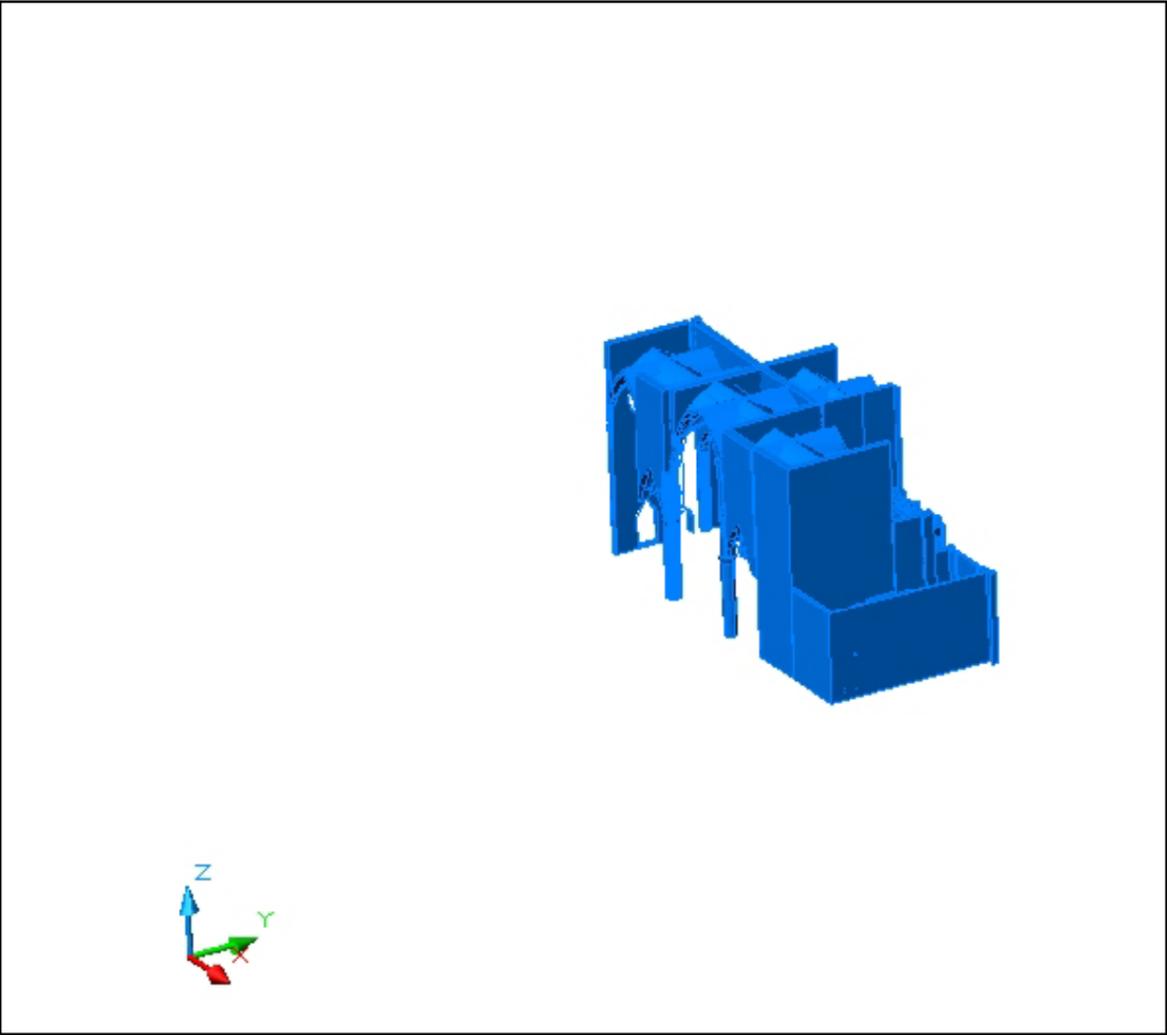
APPENDIX A

Construction phases
of the Basilica

Initial Phase: Construction of the Transept

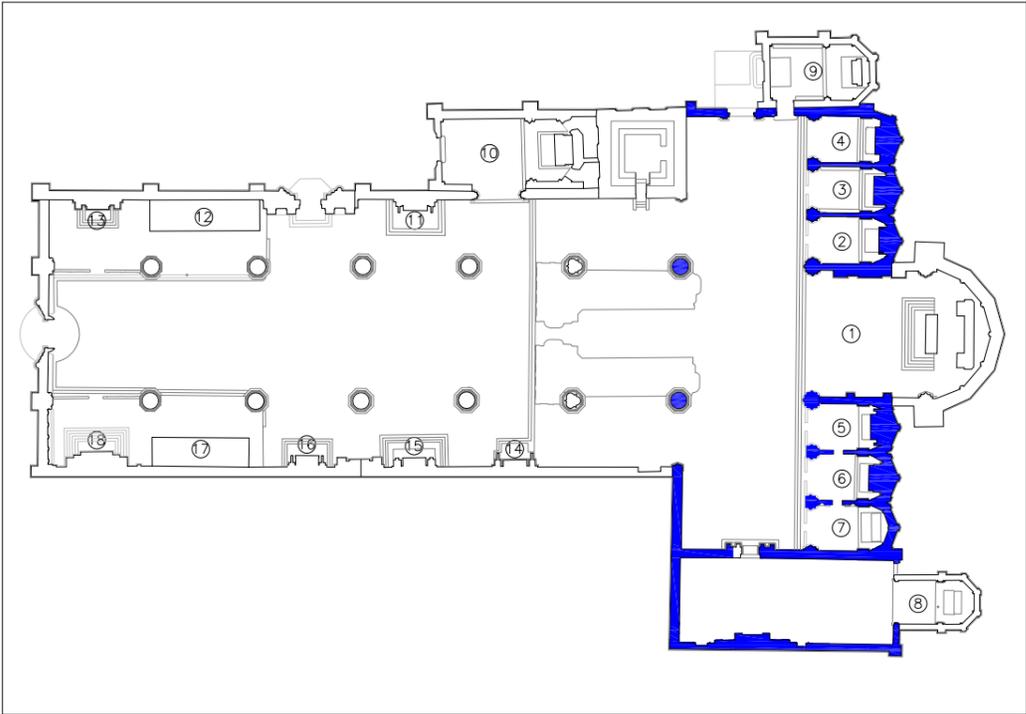


North Axonometric View



East Axonometric View

INITIAL PHASE:
1330-1361

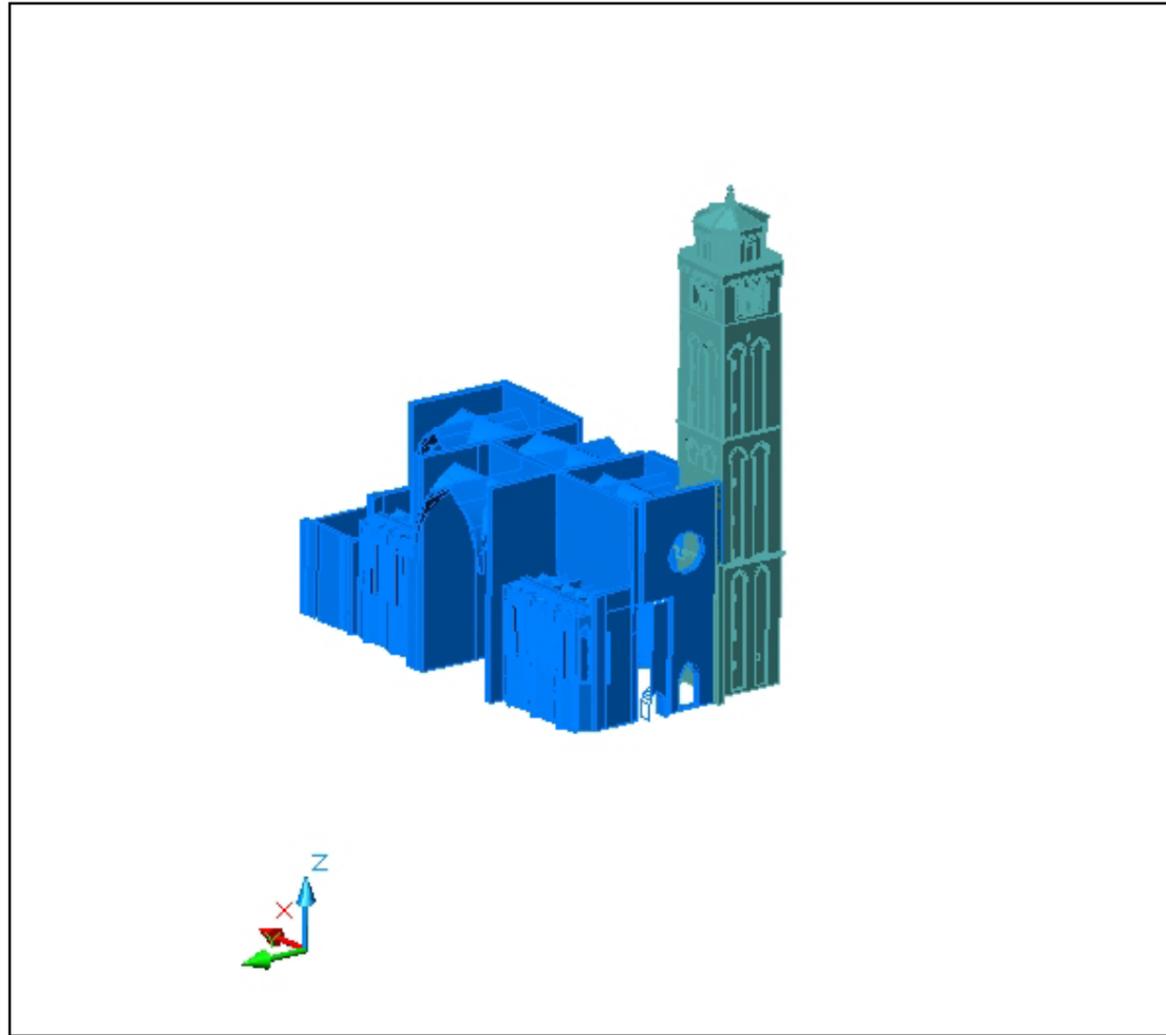


- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanese
- 5 Chapel of Fiorentini
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo

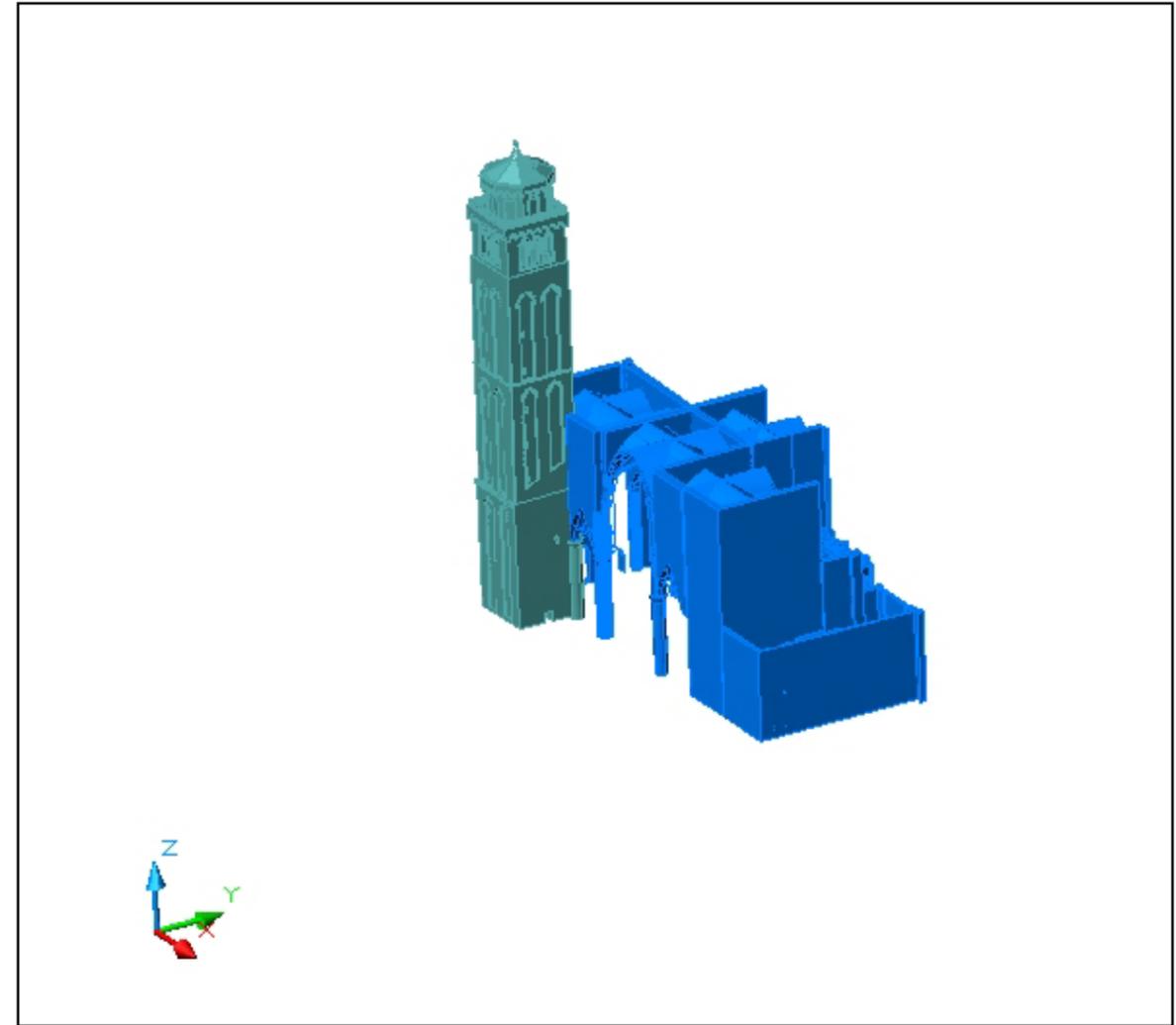
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Tower Construction

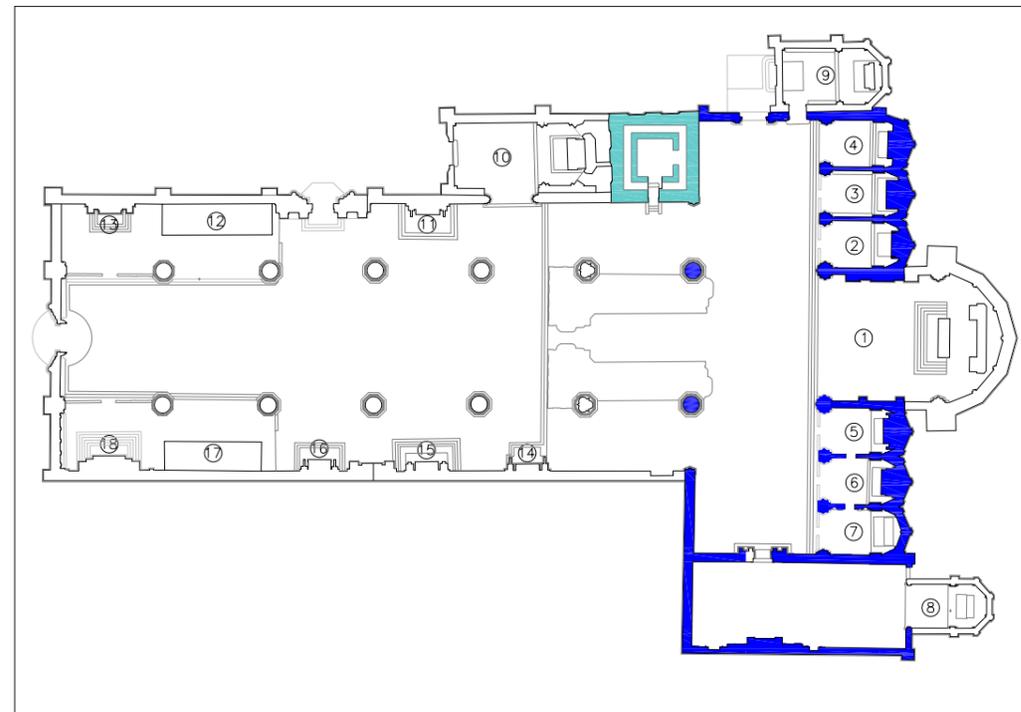


North Axonometric View



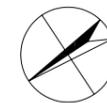
East Axonometric View

-  INITIAL PHASE:
1330-1361
-  TOWER CONSTRUCTION:
1361-1396

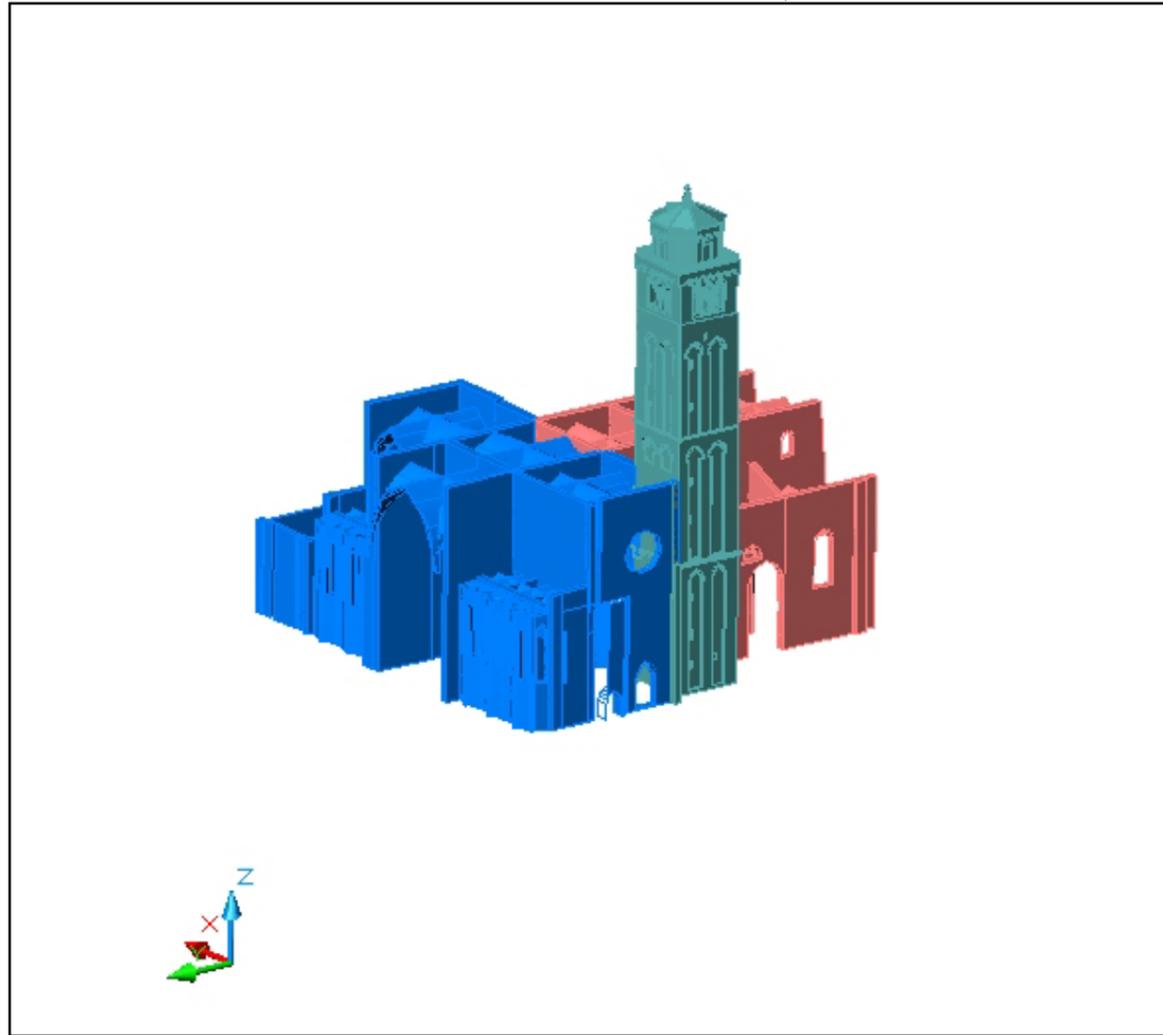


- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanese
- 5 Chapel of Fiorentini
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo

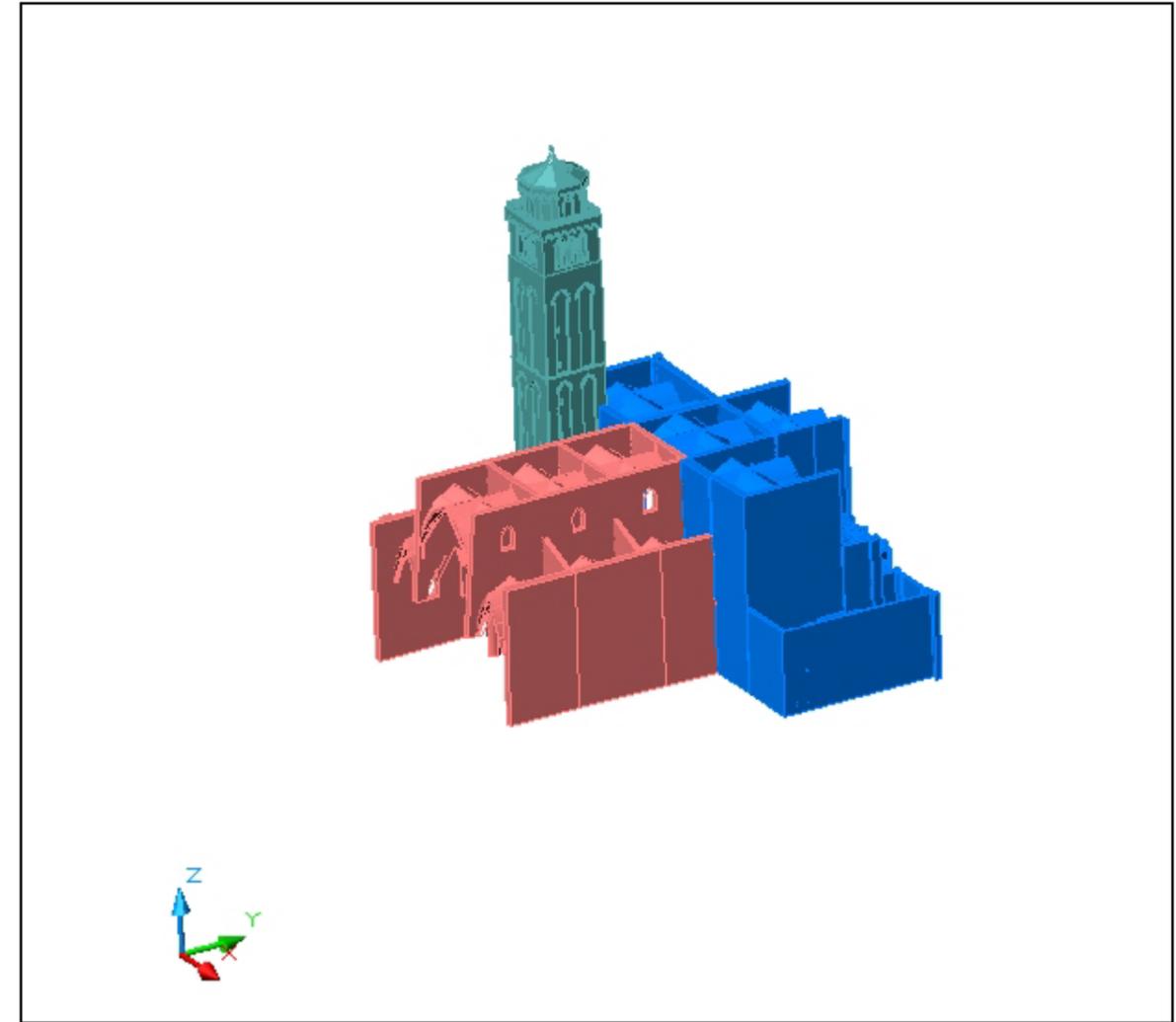
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Second Phase: Beginning of the Main Body of the Basilica

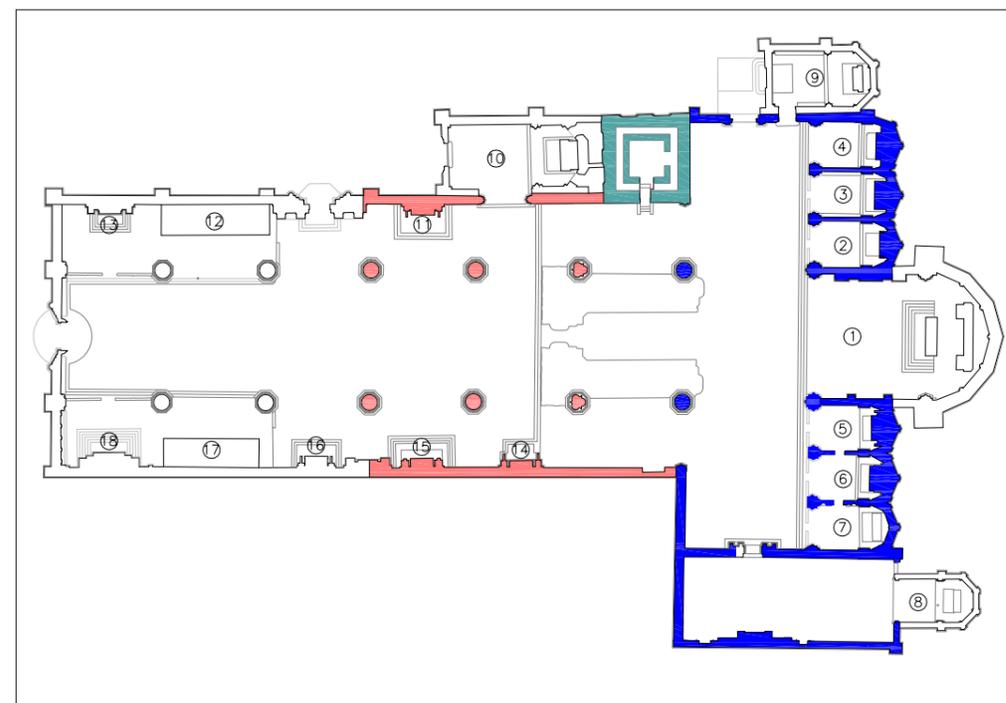


North Axonometric View



East Axonometric View

- INITIAL PHASE:
1330-1361
- TOWER CONSTRUCTION:
1361-1396
- SECOND PHASE:
1391-1420



- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanese
- 5 Chapel of Fiorentini
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo

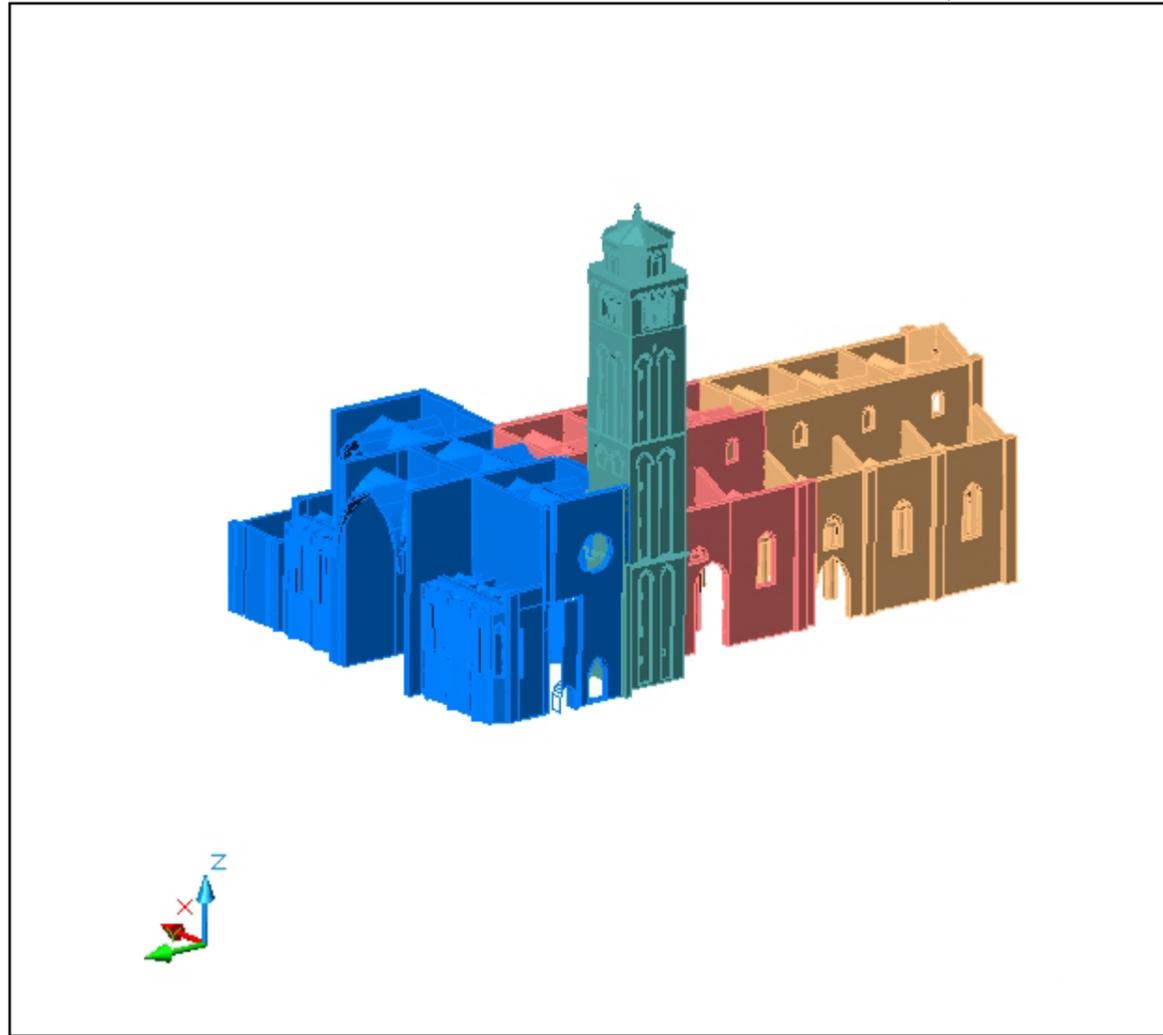
11 Pesaro Altar

14 Pesaro Altar
15 Zane Altar

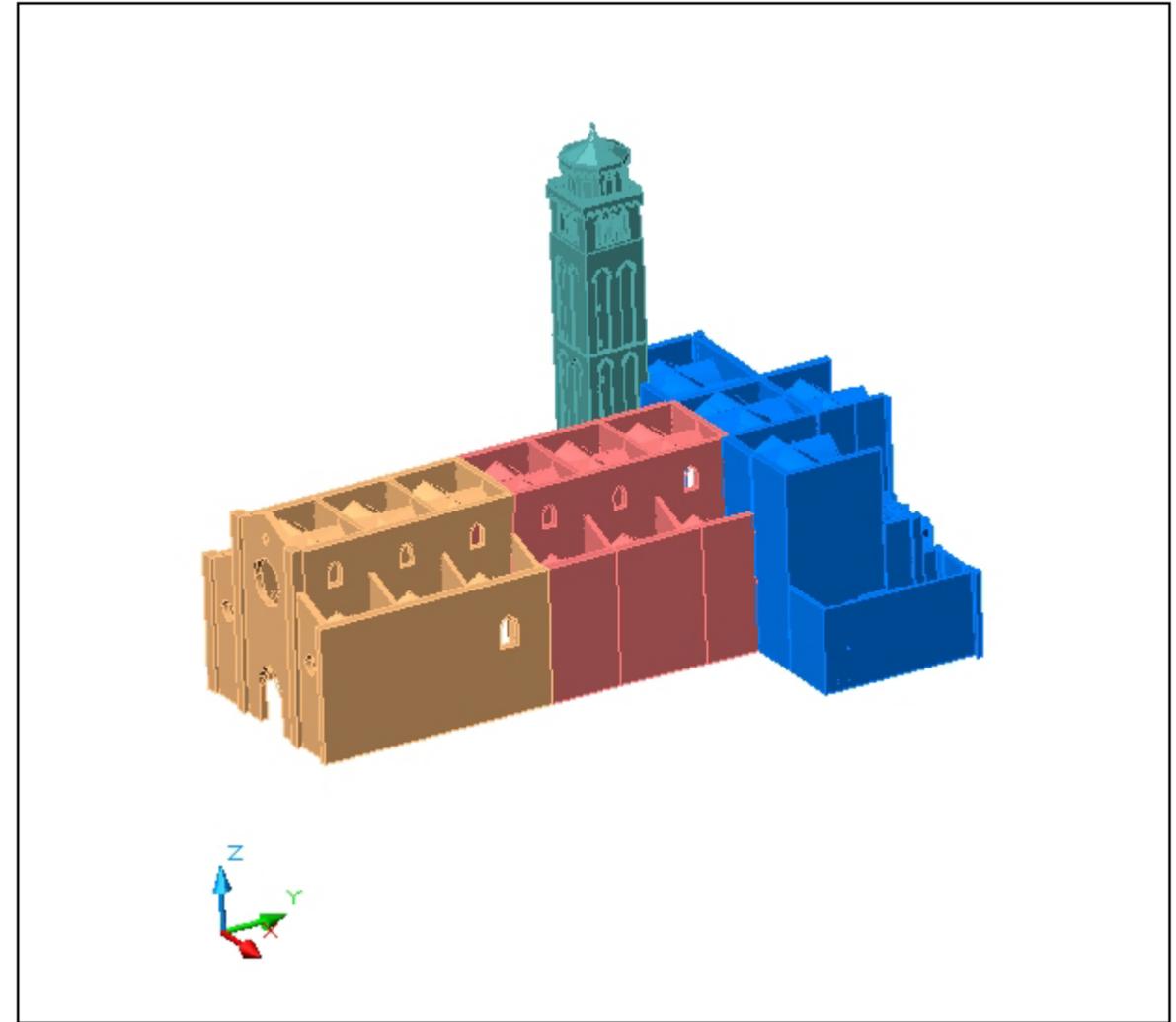
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Third Phase: Completion of the Basilica

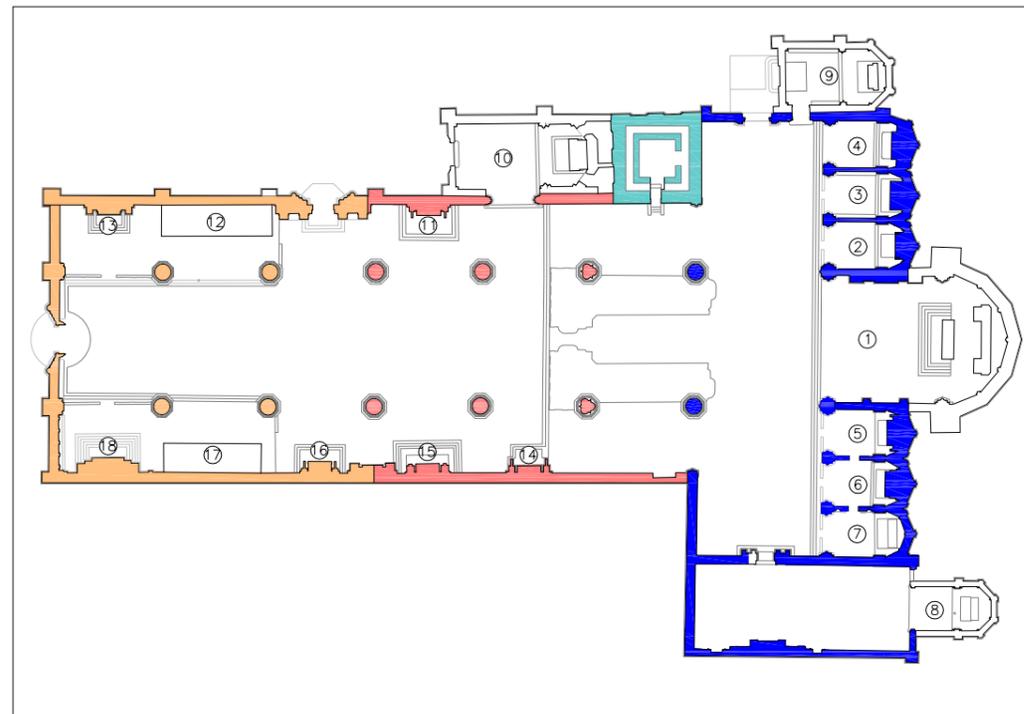


North Axonometric View



East Axonometric View

- INITIAL PHASE:
1330-1361
- TOWER CONSTRUCTION:
1361-1396
- SECOND PHASE:
1391-1420
- THIRD PHASE:
1420-1450



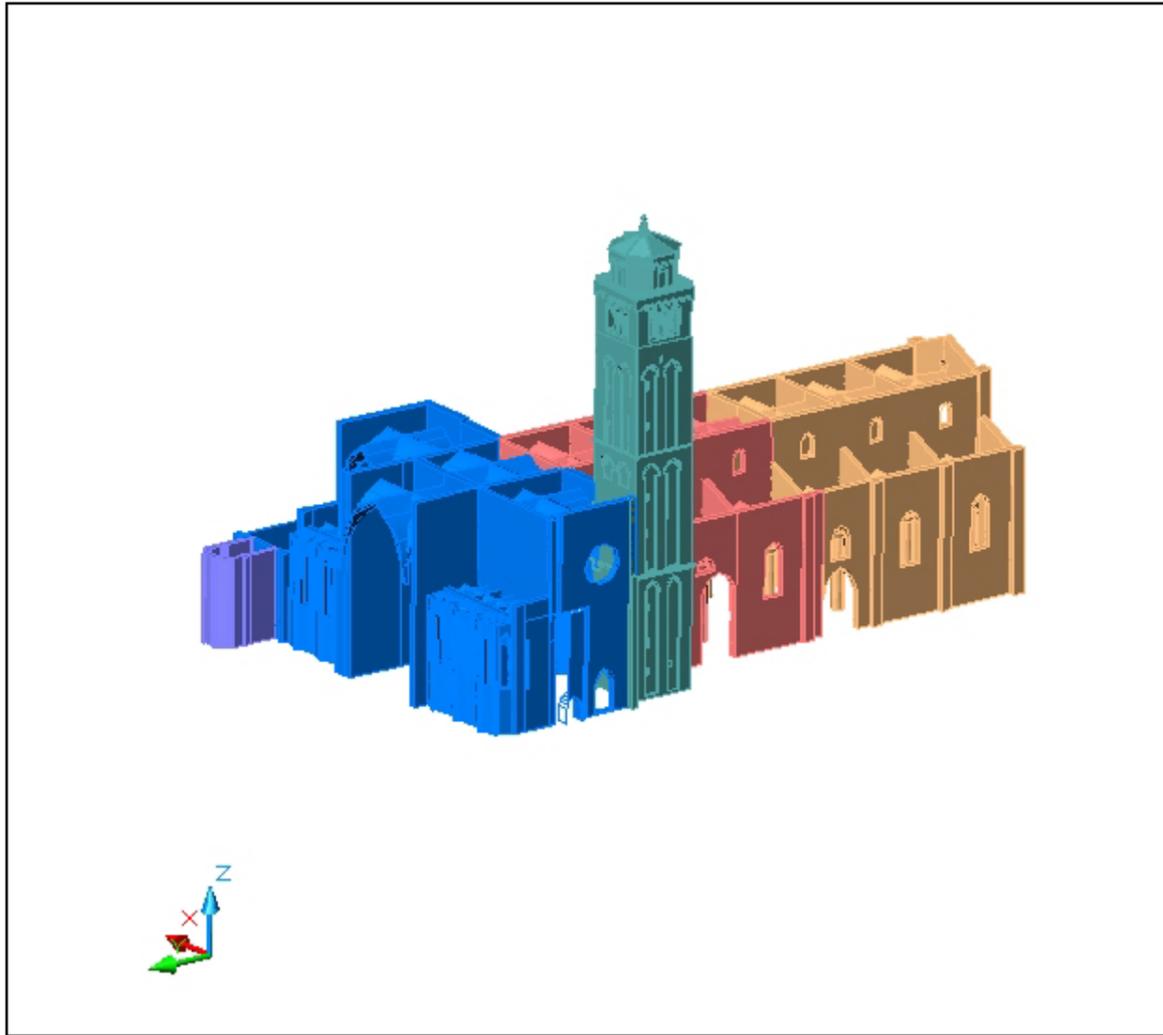
- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanese
- 5 Chapel of Florentine
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo

- 11 Pesaro Altar
- 12 Canova's Mausoleum
- 13 Altar of the Crucifix
- 14 Pesaro Altar
- 15 Zane Altar
- 16 Zane-Valier Altar
- 17 Tiziano's Mausoleum
- 18 St. Antony Altar

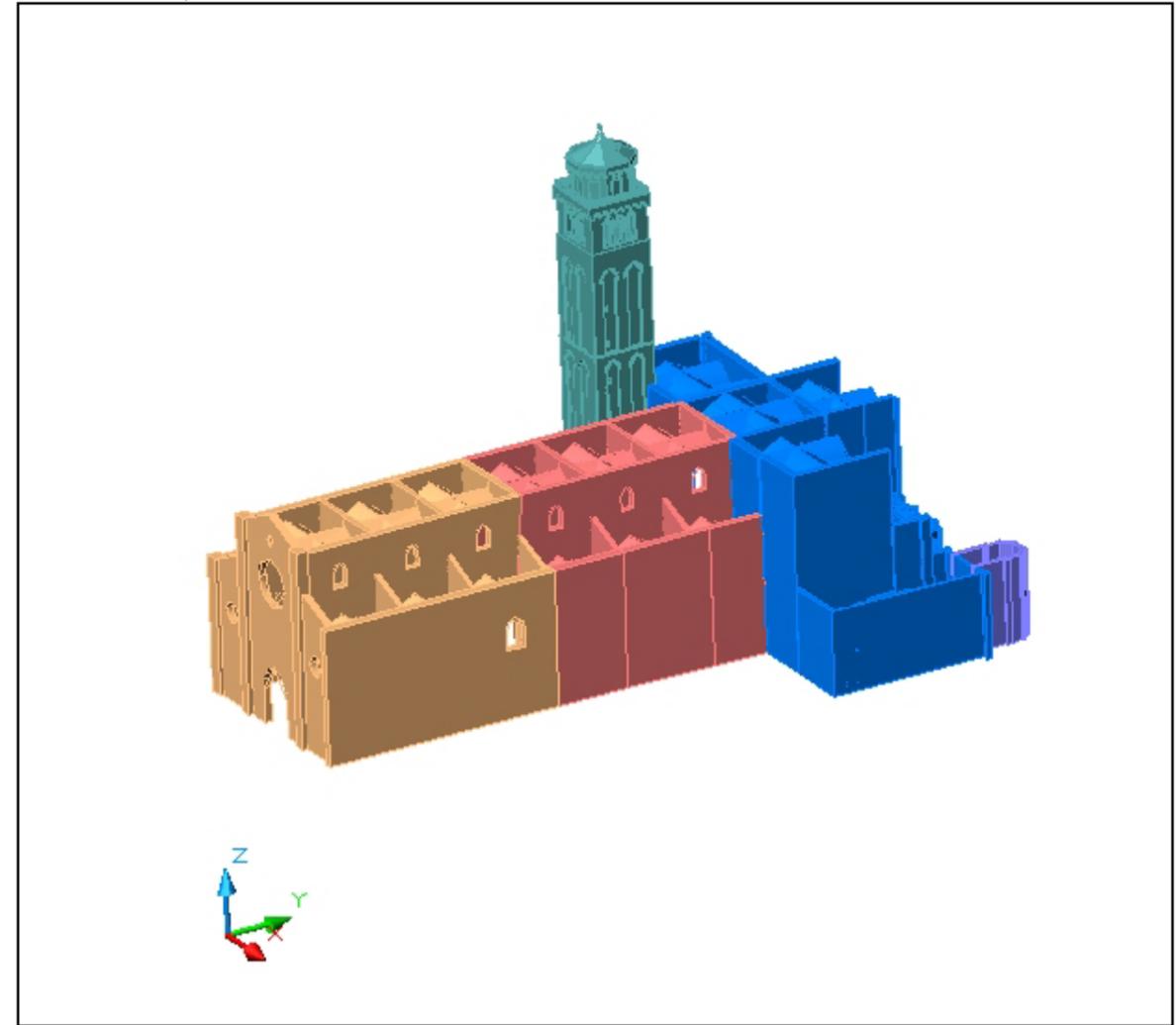
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Addition: Chapel Pesaro

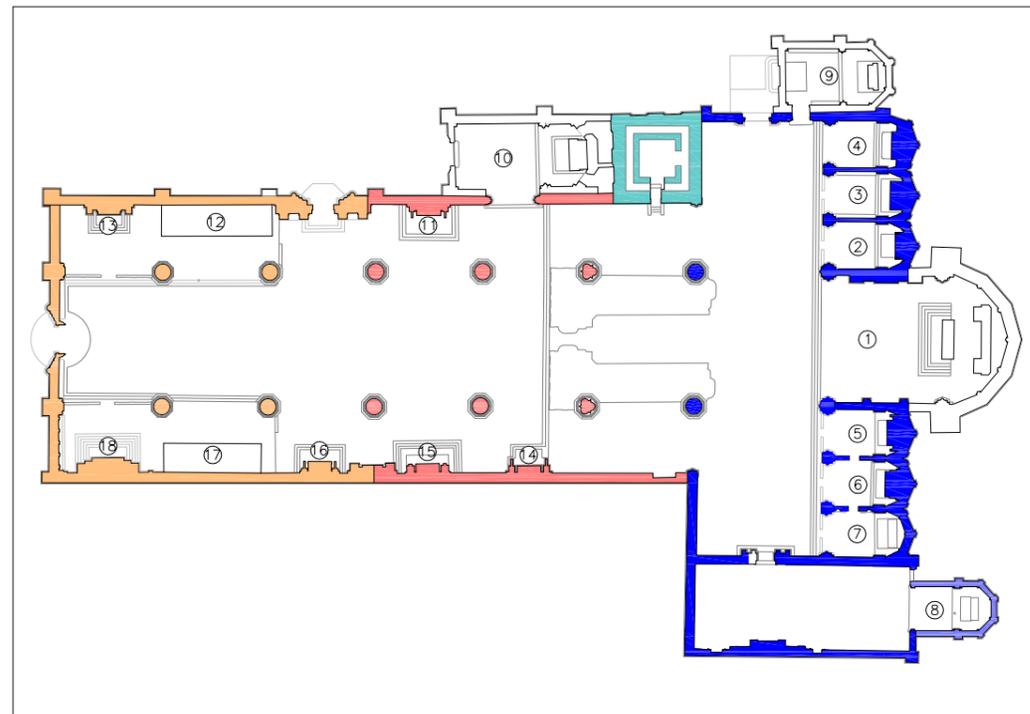


North Axonometric View



East Axonometric View

- INITIAL PHASE:
1330-1361
- TOWER CONSTRUCTION:
1361-1396
- SECOND PHASE:
1391-1420
- THIRD PHASE:
1420-1450
- ADDITION:
PESARO Chapel
2nd half XV century



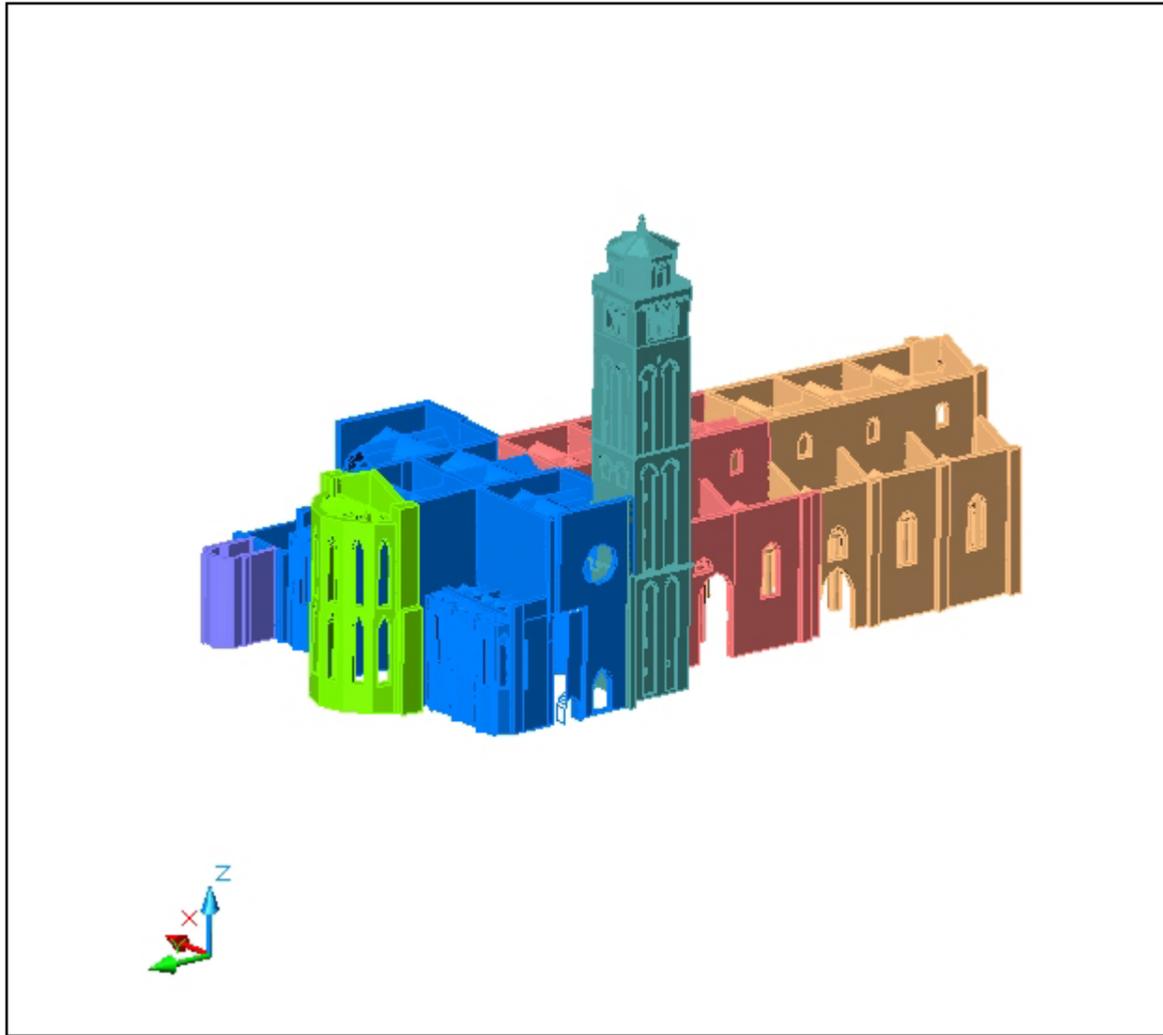
- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanese
- 5 Chapel of Florentine
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo
- 8 Chapel Pesaro

- 11 Pesaro Altar
- 12 Canova's Mausoleum
- 13 Altar of the Crucifix
- 14 Pesaro Altar
- 15 Zane Altar
- 16 Zane-Valier Altar
- 17 Tiziano's Mausoleum
- 18 St. Antony Altar

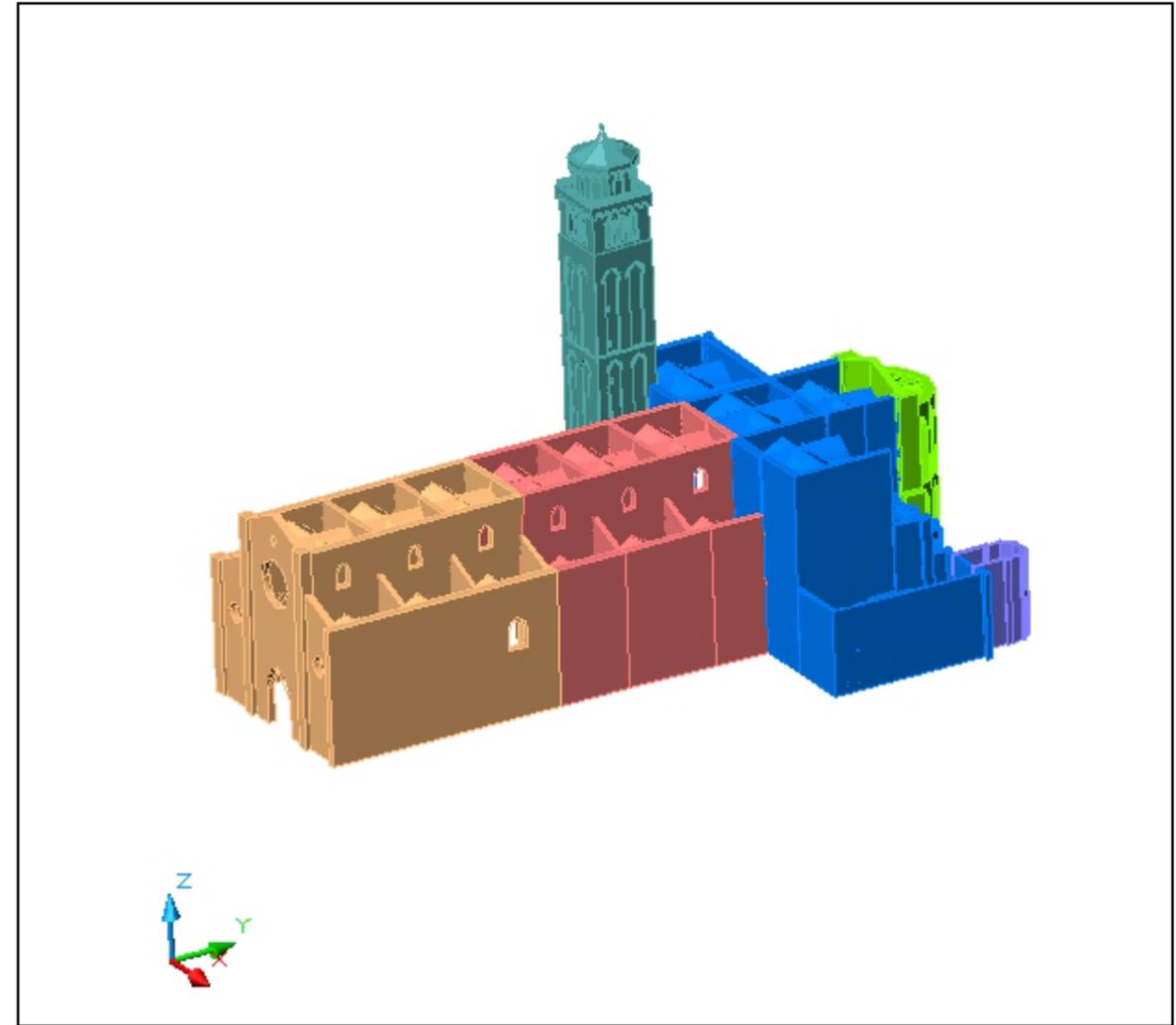
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Major Apse Construction

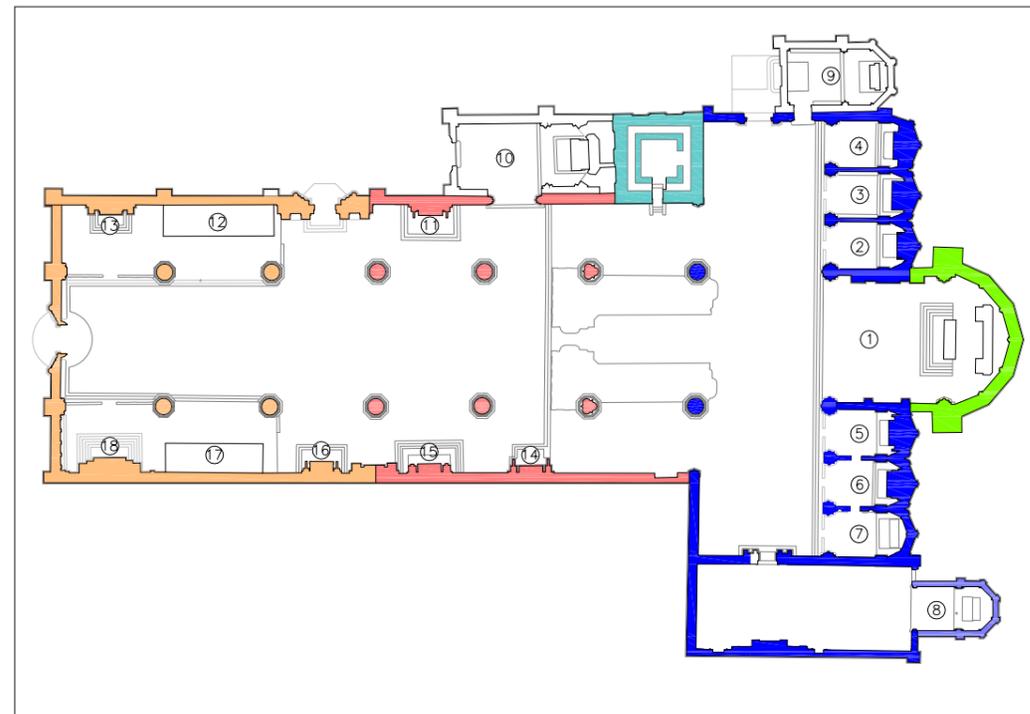


North Axonometric View



East Axonometric View

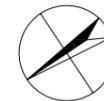
- INITIAL PHASE:
1330-1361
- TOWER CONSTRUCTION:
1361-1396
- SECOND PHASE:
1391-1420
- THIRD PHASE:
1420-1450
- ADDITION:
PESARO Chapel
2nd half XV century
- MAJOR APSE
CONSTRUCTION:
half XV century



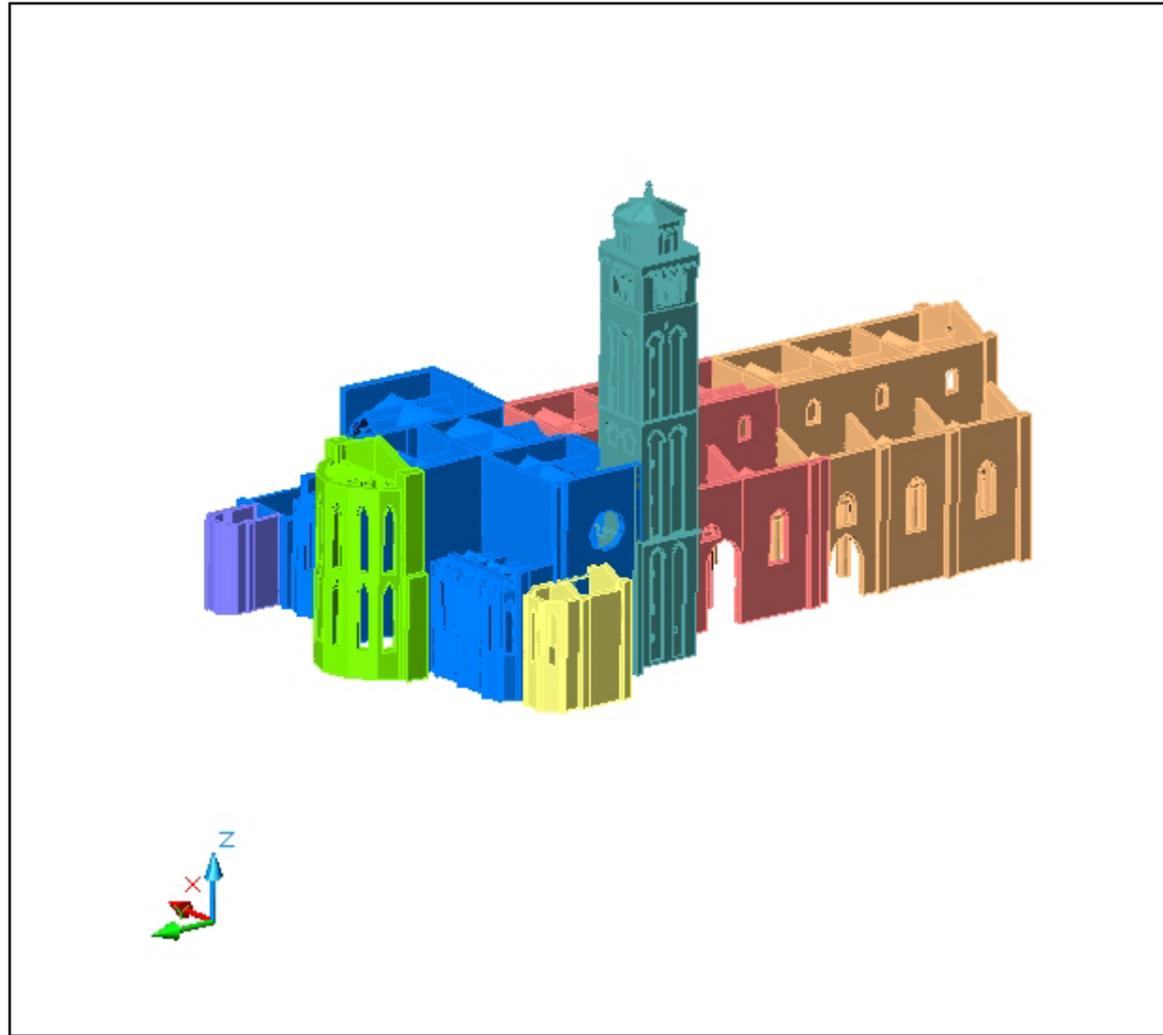
- 1 Major Chapel
- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanese
- 5 Chapel of Fiorentini
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo
- 8 Chapel Pesaro

- 11 Pesaro Altar
- 12 Canova's Mausoleum
- 13 Altar of the Crucifix
- 14 Pesaro Altar
- 15 Zane Altar
- 16 Zane-Valier Altar
- 17 Tiziano's Mausoleum
- 18 St. Antony Altar

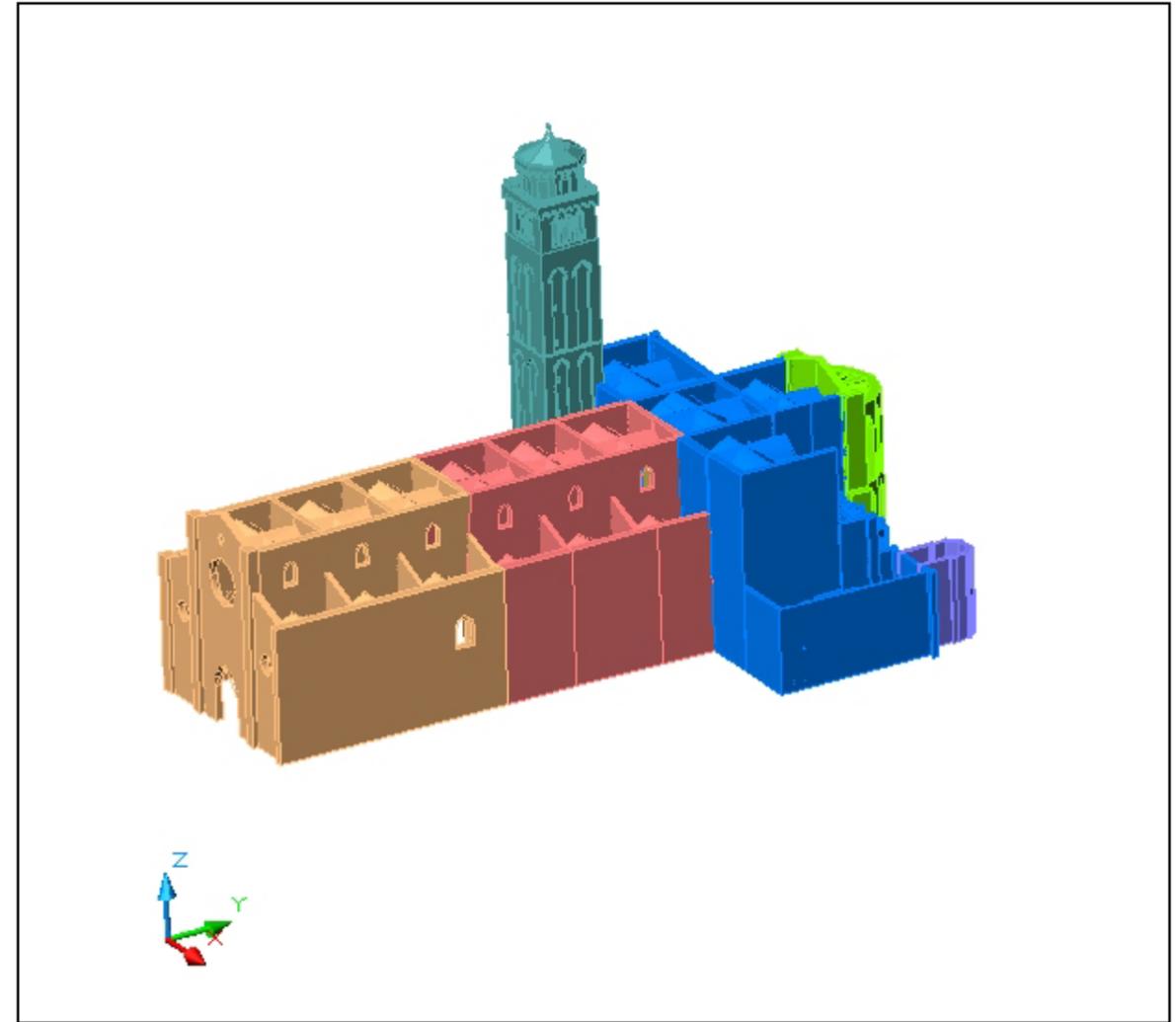
0 10 20 m



Addition: Chapel Corner or St. Mark

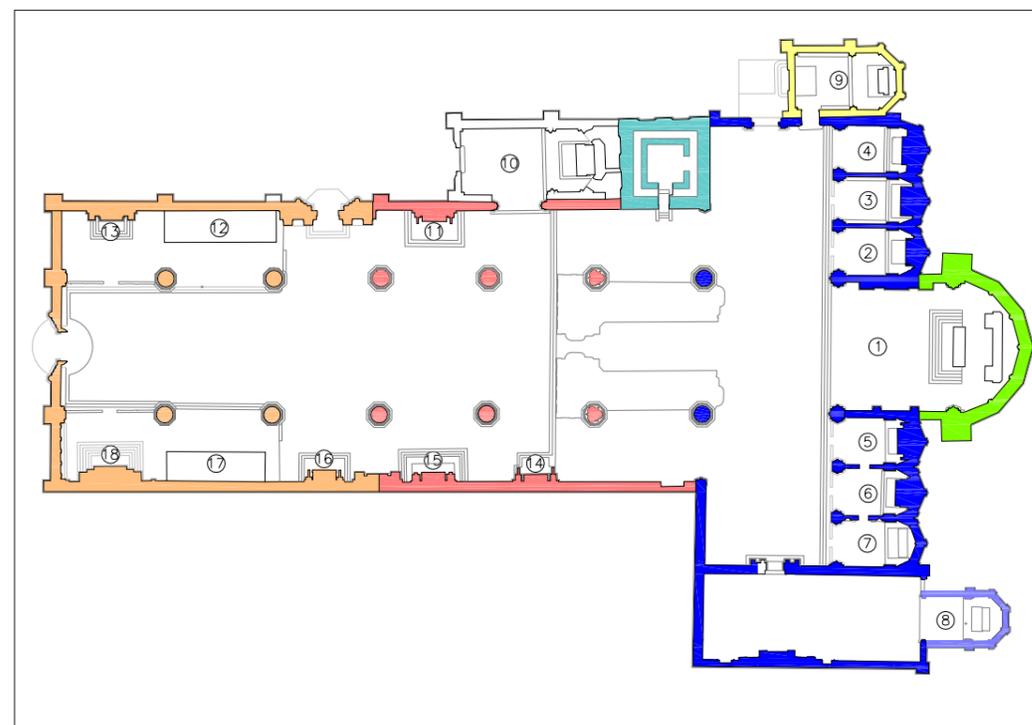


North Axonometric View



East Axonometric View

- INITIAL PHASE:
1330-1361
- TOWER CONSTRUCTION:
1361-1396
- SECOND PHASE:
1391-1420
- THIRD PHASE:
1420-1450
- ADDITION:
PESARO Chapel
2nd half XV century
- MAJOR APSE
CONSTRUCTION:
half XV century
- ADDITION:
CORNER Chapel
(St. MARK Chapel)
1417 (work begun)

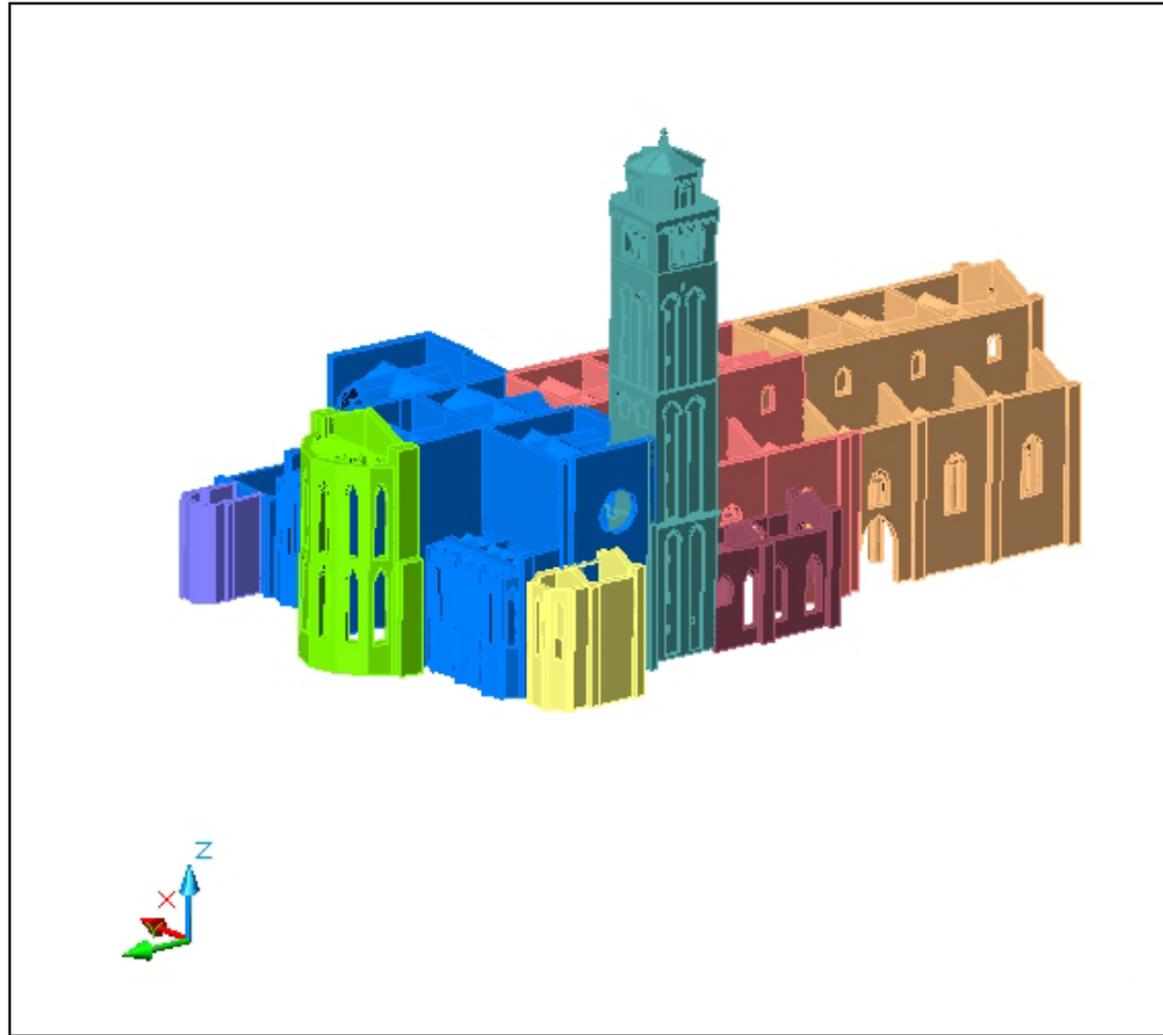


- 1 Major Chapel
- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanesi
- 5 Chapel of Fiorentini
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo
- 8 Chapel Pesaro
- 9 Chapel Corner or St. Mark
- 11 Pesaro Altar
- 12 Canova's Mausoleum
- 13 Altar of the Crucifix
- 14 Pesaro Altar
- 15 Zane Altar
- 16 Zane-Valier Altar
- 17 Tiziano's Mausoleum
- 18 St. Antony Altar

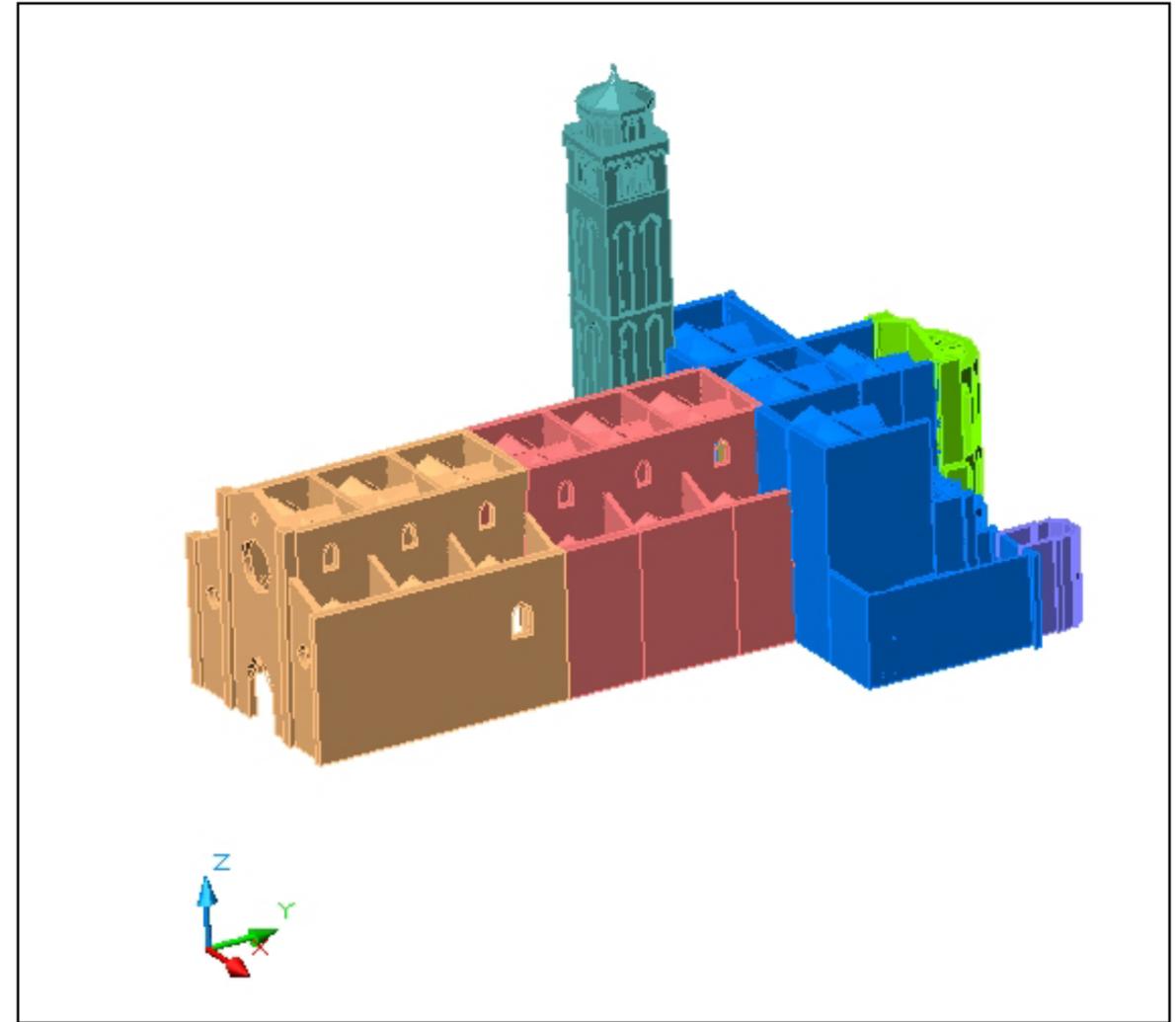
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Basilica of Santa Maria Gloriosa dei Frari

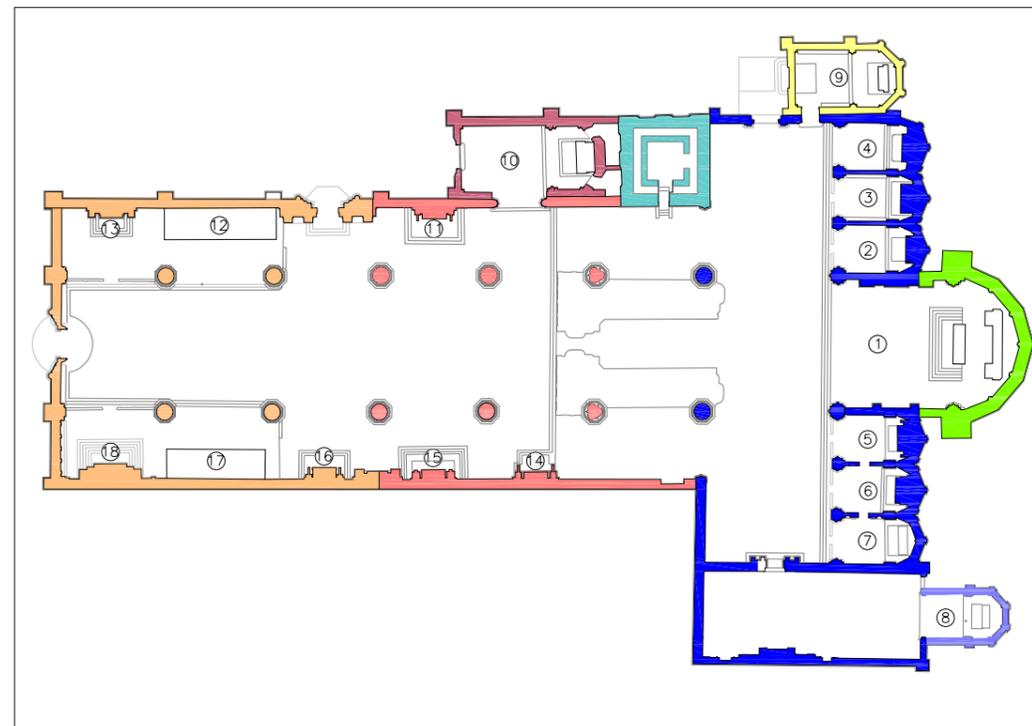


North Axonometric View



East Axonometric View

- INITIAL PHASE:
1330-1361
- TOWER CONSTRUCTION:
1361-1396
- SECOND PHASE:
1391-1420
- THIRD PHASE:
1420-1450
- ADDITION:
PESARO Chapel
2nd half XV century
- MAJOR APSE
CONSTRUCTION:
half XV century
- ADDITION:
CORNER Chapel
(St. MARK Chapel)
1417 (work begun)
- ADDITION:
MIANI Chapel
(St.PETER Chapel)
1432 (work begun)



- 1 Major Chapel
- 2 St. Francis Chapel
- 3 Chapel
- 4 Chapel of Milanesi
- 5 Chapel of Fiorentini
- 6 Chapel of Sacramento
- 7 Chapel Ca' Bernardo
- 8 Chapel Pesaro
- 9 Chapel Corner or St.Mark
- 10 Chapel Miani or St. Peter
- 11 Pesaro Altar
- 12 Canova's Mausoleum
- 13 Altar of the Crucifix
- 14 Pesaro Altar
- 15 Zane Altar
- 16 Zane-Valier Altar
- 17 Tiziano's Mausoleum
- 18 St. Antony Altar

0 10 20 m



APPENDIX B

Output data of the
Topographical survey

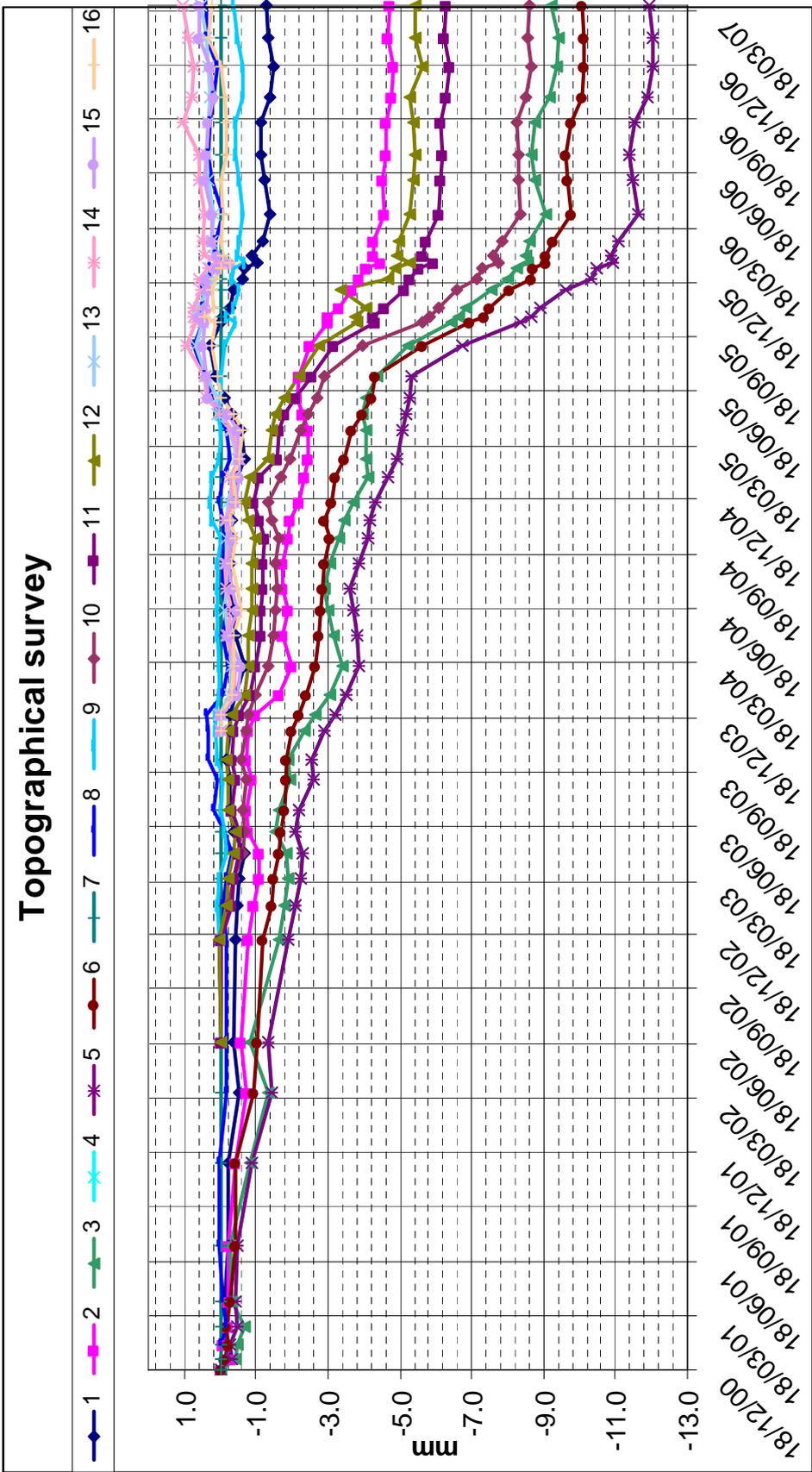
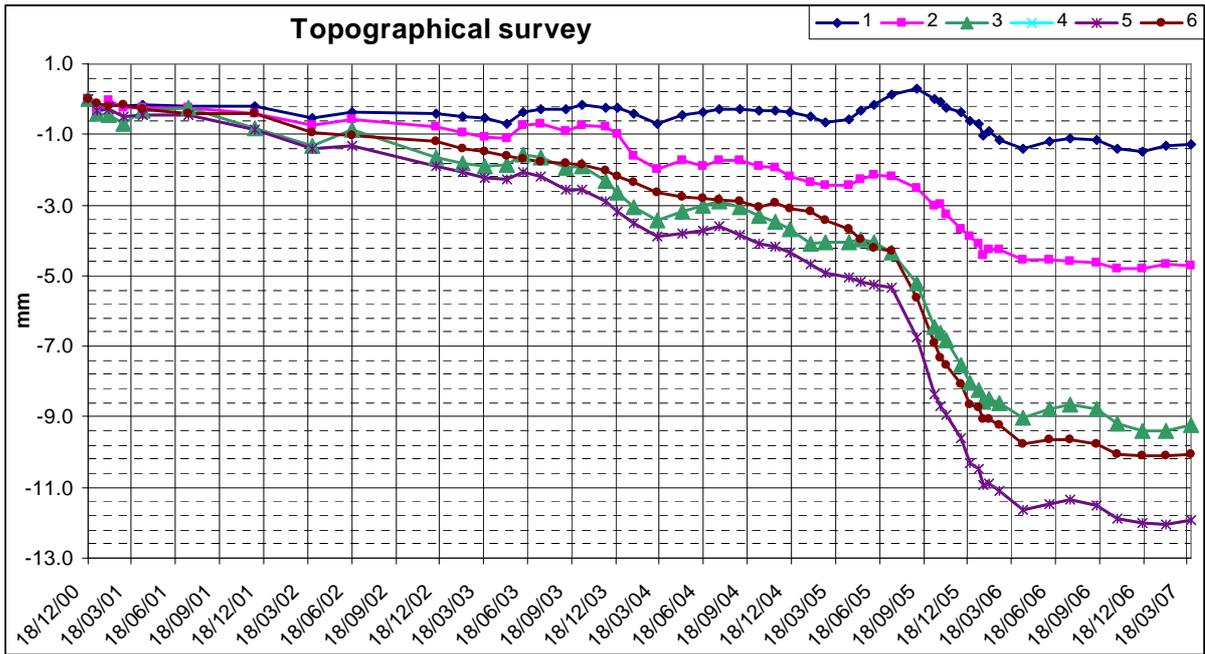
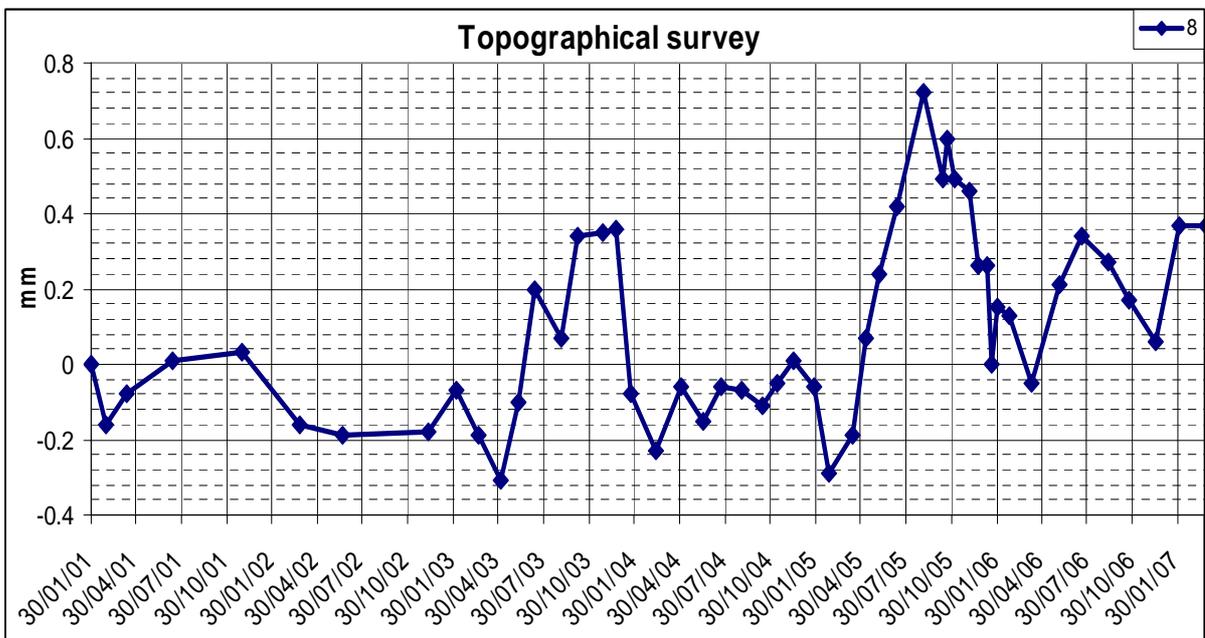


Figure B-1: Output of all the benchmarks



**Figure B-2: Output of benchmarks #1 to #6
Installed on 18/12/00**



**Figure B-3: Output of benchmark #8
Installed on 30/01/01**

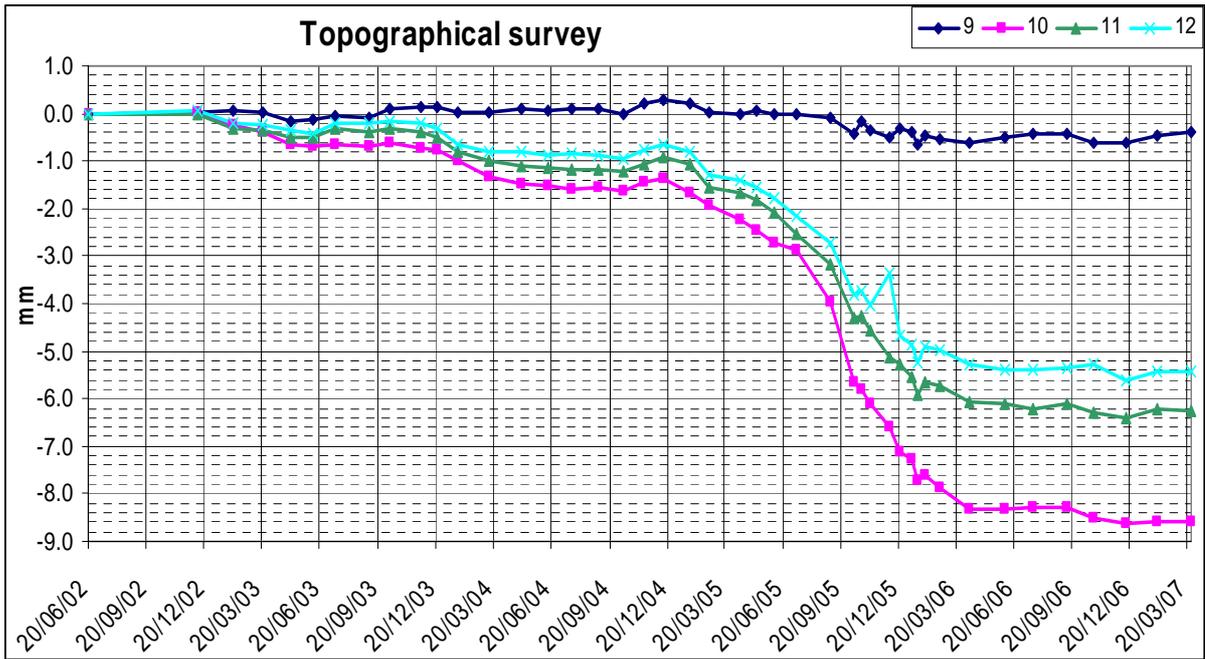


Figure B-4: Output of benchmarks #9 to #12
Installed on 20/08/02

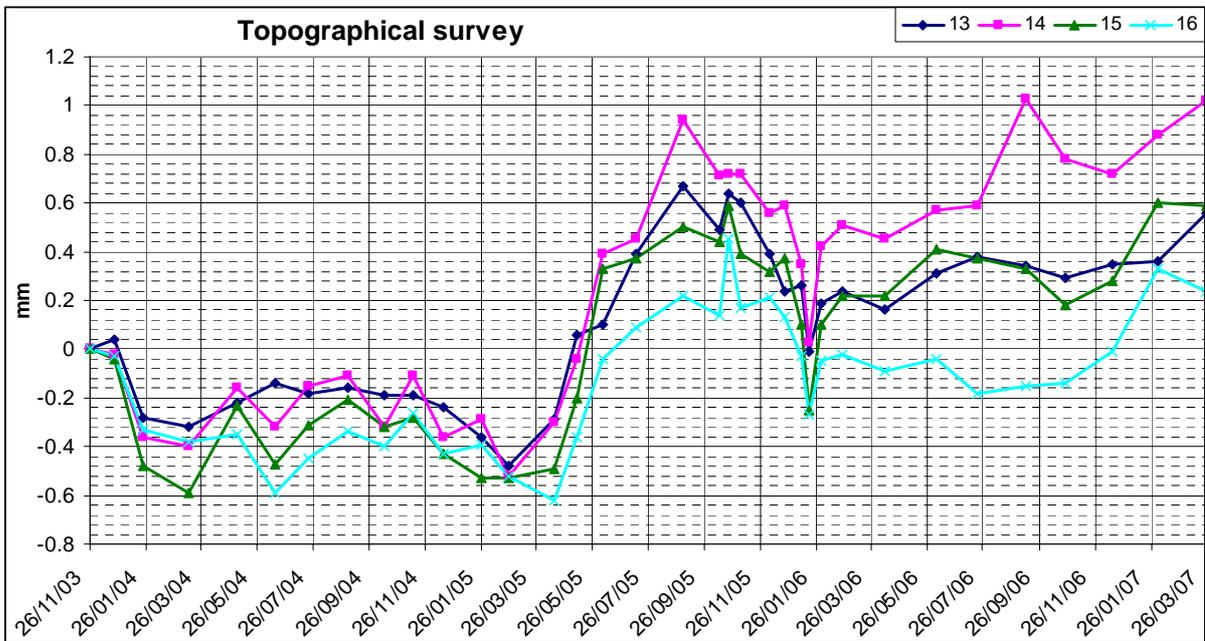


Figure B-5: Output of benchmarks #13 to #16
Installed on 26/11/03