VOLUME: 2
ISSUE: 2

DECEMBER
2021

# CULTURAL HERITAGE AND SCIENCE

ONLINE ISSN: 2757-9050
PUBLISHER: MERSİN UNIVERSITY

Cultural Heritage and Science (CUHES)

**Cultural Heritage and Science (CUHES)** is an interdisciplinary academic, refereed journal for scholars and practitioners with a common interest in heritage.

**Aims and scope**

Provide a multidisciplinary scientific overview of existing resources and modern technologies useful for the study and repair of cultural heritage and other structures. The journal will include information on history, methodology, materials, survey, inspection, non-destructive testing, analysis, diagnosis, remedial measures, and strengthening techniques.

Preservation of the architectural heritage is considered a fundamental issue in the life of modern societies. In addition to their historical interest, cultural heritage buildings are valuable because they contribute significantly to the economy by providing key attractions in a context where tourism and leisure are major industries in the 3rd millennium. The need for preserving historical constructions is thus not only a cultural requirement, but also an economic and developmental demand.

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- ✓ General criteria and methodology for study and intervention
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- ✓ Survey techniques
- ✓ Non-destructive testing, inspection, and monitoring
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- ✓ Analytical and numerical approaches
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*Volume* 2
*Issue* 2 *December, 2021*

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Cultural Heritage and Science
https://dergipark.org.tr/en/pub/cuhes
e-ISSN 2757-9050

# Structural Analysis and Reinforcement of XVI Century Building in the Center of Naples, Italy

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**Keywords**
Historical building Masonry,
Structural retrofitting,
Seismic risk assessment.

**ABSTRACT**

Most of the buildings in the historical center of the cities in Italy are quite old, and sometimes their age is more than several centuries. This means that a double activity aim should be considered by the community, the historical conservation of the cultural heritage and the static health of the structure to guarantee the safety for the people living or working in those special buildings. This double need for the "living" cultural heritage architectures sometimes arises some internal conflicts, since higher, exaggerated reinforcing interventions could lead to damage, or even cancel, some architectural evidence. The paper introduces as a case study the example of one residential historical building in Naples, Italy, reinforced considering the need to preserve the original aspect. Extensive performed diagnostic tests and specific structural numerical analysis carried out for the emblematic parts of the building are presented.

## 1. INTRODUCTION

The building we are introducing in this paper is located in Naples, south of Italy, along the "Riviera di Chiaia" street. This old Neapolitan name means in English "Riverside", and it is a long historical street facing the Gulf of Naples. Urbanization behind "Riviera di Chiaia" goes back to the end of the 17th century, when many noble palaces, including the famous Villa Pignatelli (today Diego Aragona Pignatelli Cortés museum, "Fig. 1".

The road was opened by Viceroy Luis Francisco de la Cerda y Aragón, Duke of Medinaceli in 1697, but major changes underwent in the following centuries. Originally the fancy street was opened facing the beach of the coast of Naples. Today, however, due to the intentional accumulation of ground filling a large strip of sea in the 18th century, it runs along the inner side of the Villa Comunale in Naples. This widening of the coast made "Riviera di Chiaia", created during the rehabilitation of the city, an inner street, while the seaside was replaced by "via Caracciolo", still existing today in that shape.

"Riviera di Chiaia" was actually on the beach (hence the name), and slowly, for subsequent bridging, other buildings were built on that strip that was previously the sea, advancing the coastline of about 180 m "Fig. 2".

**Figure 1.** Map of Naples 1663 – from Bastiaen Stopendaal

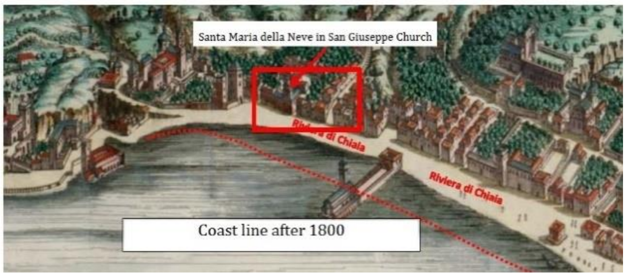


**Figure 2.** Map of Naples 1663 (detail)-from Stopendaal.

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**Cite this article (APA);**

*Wuth M S, Paterna P & Abruzzese D (2021). Structural Analysis and Reinforcement of XVI Century Building in the Center of Naples, Italy.* Cultural Heritage and Science, 2(2), 31-42

**Research Article**

**Received: 10/11/2021; Accepted: 17/12/2021**

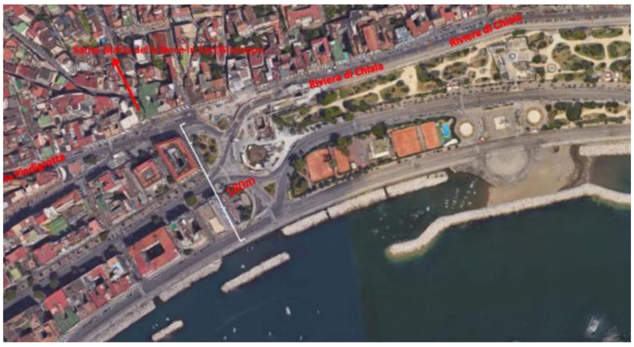


**Figure 3.** Republic Sq. and Riviera di Chiaia today (Google map in 2017)

## 1.1 Historical Presentation of the Building

The building is a relatively old historical construction. Probably it has been the first realization at the end of the 18th century or even earlier if we assume the building already existed at the time of some famous historical engravings or "view" of the 17th century or even earlier. The buildings are facing the "Riviera di Chiaia", but even though its age and its ancient plan, it has been remodeled several times. This is confirmed not only by the most recent plants (1775) as that of "Giovanni Carafa", Duke of Noia, but also from previous plans and views.

In the plan "Fig. 4" of the "Duca di Noia" we can see a building located in the next block on the side of "Santa Maria della Neve" church (we can recognize the church in the plan). In fact, there are clear signs of expansion of the building volume, with a first elevation probably built in the mid-nineteenth century, with techniques similar to the original ones, and a second elevation, built with a reinforced concrete frame system with beams and columns, probably built between the 1950 and 1960 ("Fig. 8").


**Figure 4.** Plan of Giovanni Carafa, Duca di Noia, 1775 (detail of the Riviera di Chiaia).


**Figure 5.** Naples-1632. Map-view by Alessandro Baratta


**Figure 6.** Map of J. Stockdale 1800 (detail)

**Figure 7.** Google map 2018. Building with green roof

## 1.2 Comment from Visual Building Condition Assessment

In the figures we can see that the decoration of frames in the window from the first floor up to the third floor are the same frames and finished with stucco decoration on the third floor. On the other hand, on the fourth floor in the very simple window without frames, it means that this floor was built at a different time from the floor below, perhaps in the year 1900 because of the construction technician of this floor still in masonry. After that, the top floor is recently built, because it is built in reinforced concrete.

**Figure 8.** The facade of the building. In red the two floors probably added during the last two centuries

Historic masonry structures have been exposed to long- and short-term influence of various types of decay, which cause material deterioration and structural resistance reduction. Severe levels of material deterioration and degradations can cause structural failures in time if they constantly take place on critical structural elements. A historical building that has been evaluated as "structurally safe" according to a visual structural assessment survey might be at risk because of the hidden stress of the material.

Among the anthropic damaging activity, we can remind the construction (2000-2015) of a new subway tunnel 15m in distance from the building and 20m in depth. On block more to the east, the "Guevara di Bovino" building (17th century) partially collapsed on March 4, 2013. "Fig. 9".

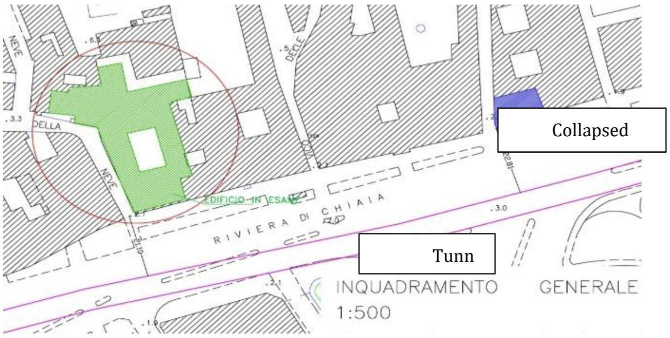


**Figure 9.** Map of the building, the tunnel line in construction and the collapsed building.

**Figure 10.** Neighboring building partially collapsed in 2013. Next to it is the construction site of the metro station

## 2. PRELIMINARY STUDY and METHOD to IMPROVE the STATIC and SEISMIC RESISTANCE of the BUILDING

To evaluate the existing static and seismic resistance of the building, some preliminary analysis and diagnosis have been carried out. In the structural analysis this activity is recognized as very important to improve the knowledge of the structure since most of the time-specific documents and information are missed. It means original design drawings, material characteristics, design loads, construction techniques. Then a geological-geotechnical campaign has been carried out, including a hydrological investigation of the water table.

Then four main structural elements, emblematic for the behavior of the building, have been deeply studied. The structural masonry element analyzed is the double-cross vault on the ground floor, the double sail vault on the first floor (in the room overlooking the courtyard), the rampant Roman staircase (quite traditional in same age buildings in Naples), the main façade at overturning risk. Numerical analyses have been performed, as well as some on-site mechanical tests.

### 2.1 Structural and Geotechnical Tests

To investigate the mechanical properties of the existing main masonry single and double jacks test have been carried out as the following: n. 2 tests with single flat jack (ASTM C1197/2004) and n. 1 test with single and double flat jack (ASTM C1197/2004). Two tests were carried out on a single flat jack (named n. 1 and n. 2 ) and located on the first floor and one test with a single and double flat jack (named n. 3) on the ground floor. The tests were conducted following ASTM C1197/2004 "Standard test method for in situ measurements of masonry deformability properties using the flat jack method".

The expected error on this type of test is in the order of 0.02 MPa for pressures (about 0.2 Kg/cm2) and about 0.001 mm based on the measurement of 300 mm.

Tests with single flat jacks allow determining the stress state existing in the masonry, by reading the pressure necessary to balance the convergence of the edges of a cut made perpendicular to the test surface and in which a flat jack is inserted, controlled by pump hydraulics. The displacements induced by the deformation are measured using a high-precision digital comparator on check plates previously fixed to the wall. The test with double flat jack, on the other hand, is based on the determination of the behavior of a masonry subject to monoaxial compressive stress induced by the insertion of two flat jack in two parallel cuts in the masonry, perpendicularly to the direction of measurement. The deformations of the masonry segment are induced by the load of the jack. "Fig. 12" will give information about the Young elastic modulus of the masonry and even its resistance, if the test reaches the local collapse of the material.

**Figure 11.** Flat jack test on the ground floor wall

Single jacks and double tests, carried out on the first floor and ground floor walls, gave an evaluation of the maximum stress of the masonry due to the vertical loads.

#### 2.1.1 Test with Single Flat jack - MP3D

Table 1. Elastic modulus of Young

| Analyzed interval | Pressure MPa | Vertical deformation | Vertical Elastic Modulus MPa |
|---|---|---|---|
| 3-2 | 0.19 | 2.79E-04 | 681,76 |
| 4-2 | 0.38 | 4.56E-04 | 833,81 |

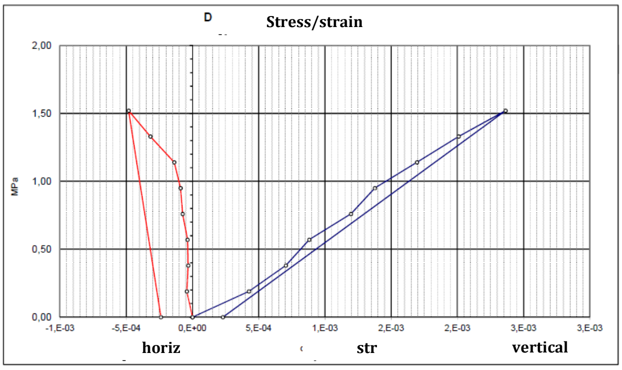


**Figure 12.** Test with flat jacks carried out on the ground floor wall

On the first floor, tests were carried out with a single jack on two types of masonry:

a) continuous masonry in squared blocks of yellow tuff with thin layers of mortar in between;

b) continuous masonry in solid bricks laid with thin courses of mortar. In both cases, the measured operating pressure was around 0.7 MPa; the breaking point was evaluated to be at least greater than 1.5 MPa.

These data are compatible with the structural modeling carried out for the building, in which the loads reported floor by floor up to the ground floor were considered.

**Figure 13.** The main facade on the Riviera di Chiaia. In the red frame the last added floor made by r.c.

### 2.1.2. Geognostic Surveys

A seasonal (six months) campaign has been carried out, with boreholes, water table control, inclinometers, Masw geoelectric test.

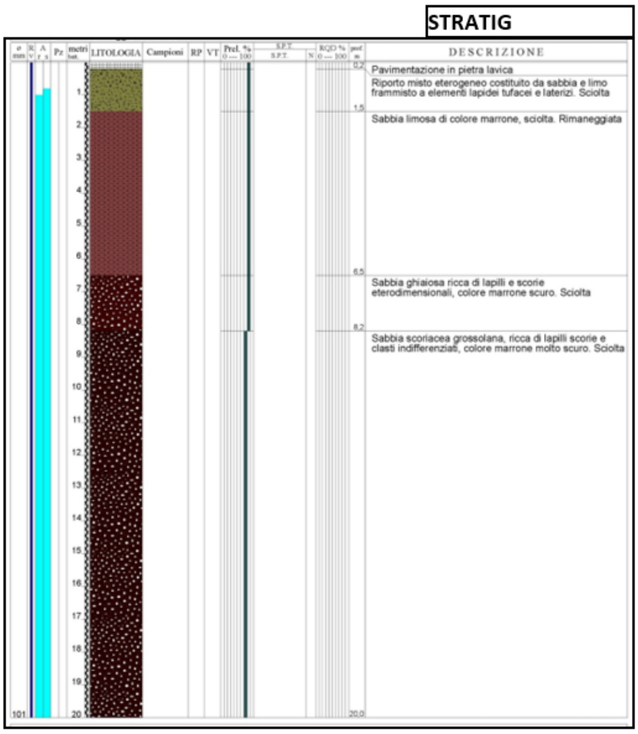


**Figure 14.** Stratigraphic column at the borehole S3

Stratigraphic column S3 (in the hole subsequently used for the inclinometer). It is evident the presence, expected, of sands and silts up to a depth of about 6.5 m from the ground level.

The borehole S3 after extracting the coring from the soil was equipped with a settling-inclinometer tube and monitored for azimuth movements.

### 2.1.3. Control of the Water Table

The surveys were carried out for six months, during which an alternating behavior of the groundwater is plausible, for instance in conjunction with weather events, but also correspondence with any effects of, albeit modest, tides, and finally thermal variations. The hypothesis of a silty-clayey sub-layer, susceptible of short or medium-term response to changes in the water table, is confirmed. This means that the building, like many other buildings overlooking the Riviera di Chiaia, is subject to slow movements in the foundation which also affect the structure of the building itself. It is also true that the age of the building (hundreds of years) should have reduced this effect, and therefore the most recent movements can not be entirely correlated to this condition of variation of the groundwater table "Fig.15".

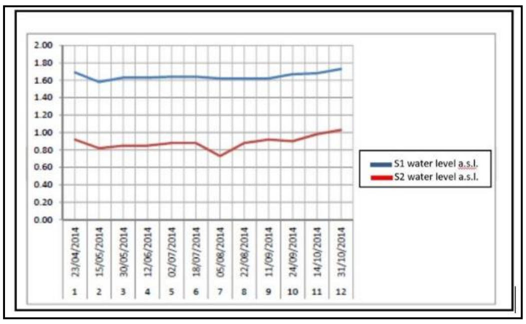


**Figure 15.** Piezometric control of the water table

### 2.1.4. Inclinometric/settling Measurements

From the inclinometer we could get the following information:

a. a deformation of the drilled pipe out of the lead of about 285 mm;

b. a deformation trend represented by the graphs shown below;

Indeed the analysis of the data collected in the observation period studied, after an initial evolutionary trend in even minimal movement, corresponding to a settling of the measuring system, especially in the most superficial portion of the tube, it has not undergone further movements except within the range of less than 1.5 mm, which can be classified as part of the instrumental error.

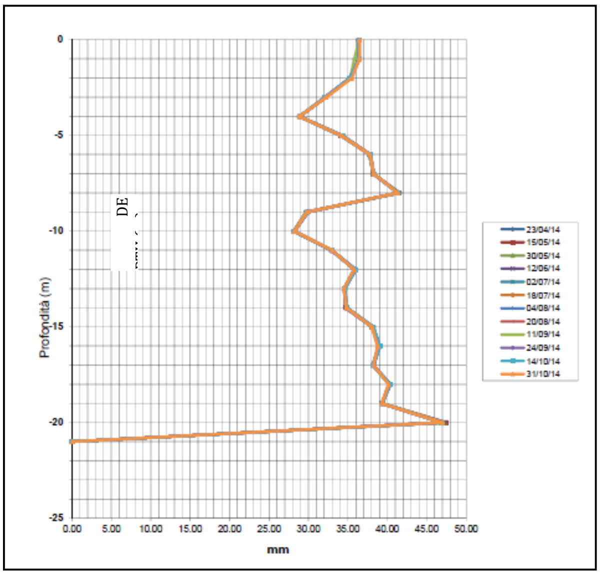


**Figure 16.** Horizontal displacement of soil borehole according to the inclinometer

### 2.1.5 Surveys with Masw Method

The results from the MASW method test (Multichannel Analysis of Surface Waves) gave the following graph, giving information about the soil density according to the depth and to the measured shear wave velocity.

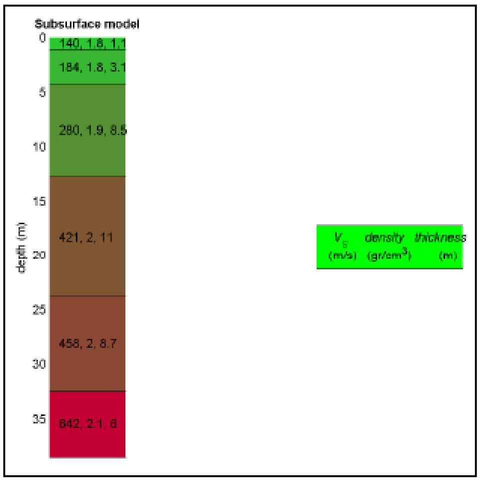


**Figure 17.** Soil density – one of the three MASW test

### 2.2. Cracking Map

The first phase of investigations, started on 22 July 2011, in some rooms on the first and second floor of the building showed some vertical and horizontal cracks on the main structural wall, some of which probably related to previous settlement movements, dating back over ten or twenty years) and the lack of horizontality in some areas of the first floor.

During this phase the geometry of all the accessible areas of the building and the cracking state were carried out, which led to the production of drawings containing plans, sections, and elevations of the building, in which the crack pattern was found during the survey operations were also reported. The cracks were then classified according to 3 levels of severity, gradually increasing: *capillary crack*, less than 1 mm width, *medium crack* of about 1mm in size*, evident crack* of more than 1 mm, but less than 1 cm, certainly passing through the wall ("*penetrable with a knife blade*"). The cracks were best represented, in plan, section, and elevation, looking for, where possible, to clarify the kinematics of the cracking, connecting the horizontal and vertical pattern "Fig. 18, Fig.19".

The cracks and lesions found in the structural elements and due to recent movements led to the hypothesis of an increase of the loads. Furthermore, the non-horizontality of some floors or rooms can lead to the assumption that there has been foundational subsidence, now stabilized, with limited excursion due to the water table. The vertical cracks in some connection between two orthogonal walls lead to fear that the building is going to open toward the main street, at the main façade. The other conclusion after the flat jack measurements is that the lower part of the main wall, on the ground floor, is subjected to loads so high as to begin an irreversible phase of crushing due to unsustainable vertical load.

Another element that should not be underestimated is the presence of cracks on the intrados of some arches on the first floor, in the middle plane of the axis of the arch, which signal an overload on the upper floors, as well as on the large arch on the ground floor, near the doorman office.

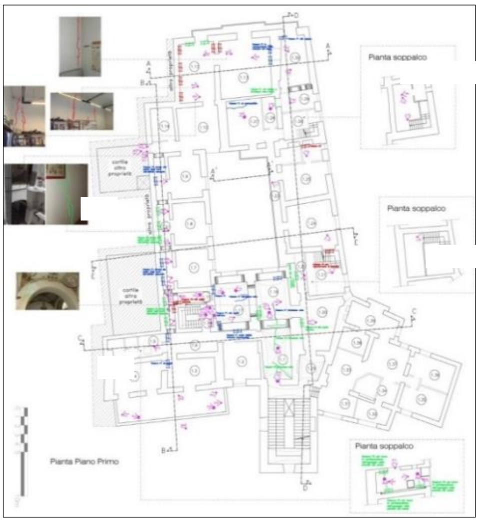


**Figure 18.** Cracking map at the first floor

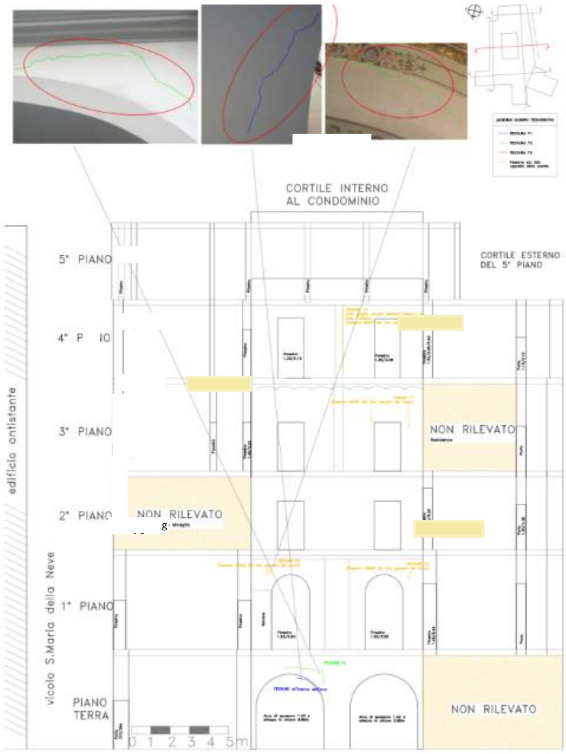


**Figure 19.** Crack map on the vertical walls in transversal cross-section E-E

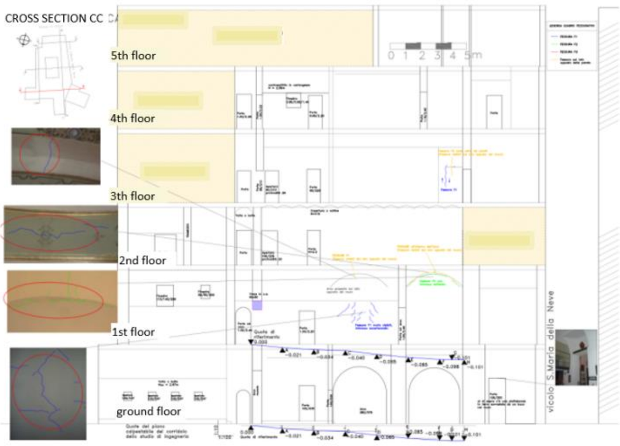


**Figure 20.** Crack map on the vertical wall in transversal cross-section C-C

**Figure 21.** Crack on the corner of the main façade

## 2.3. Structural Analysis of Emblematic Elements

Here we introduce the first structural analysis of the building, related to the cross vault on the ground floor. The analysis of the masonry cross vault on the ground floor considers the vertical loads, and the approach is similar to that one for a masonry arch.

**Figure 22.** The cross vault on the ground floor, in the hall

As usual, the state of minimum thrust is configured as the state of natural equilibrium in the masonry cross vault. Among all the kinematically admissible thrusts, the one corresponding to the minimum thrust corresponds to the limit thrust. But since we are not going to solve this problem by kinematic approach, we assume a non-linear finite element model where the material masonry has the following properties:

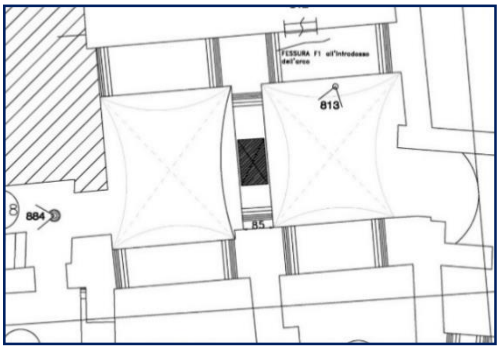


**Figure 23.** The plan of the cross vault

Elastic modulus = 3000 MPa
Unit weight = 16 kN/m^3

The collapse criterion is connected to the typical Mohr-Coulomb model for the behavior of the masonry. The masonry is considered with a limited compressive resistance of the material equal to 2.5 MPa, while the limited tensile stress is about 1/12 of that one for compression, according to the double flat jack test curve and to the classification of the masonry.

Then an increasing live load has been assigned to the structure until the collapse. From the results we could get the ultimate load for the structure, and then the safety factor again the operating load.

After performing a non-linear incremental analysis (pushover) for vertical load, we obtain the curve as in “Fig. 24”.

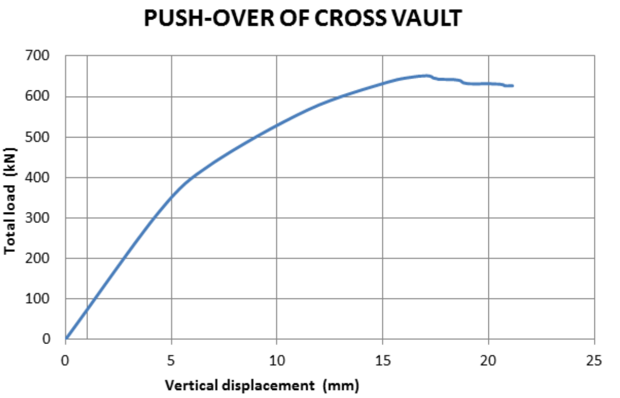

**Figure 24.** Result from vertical push over for cross vault

The results of the non-linear analysis give an ultimate load of 650 kN, while the operating load is about 583 kN. Then the safety factor is as follows:

η = 650 kN/583 kN = 1.12 > 1

We can conclude that the cross vault twin has a very limited structure safety factor, and probably some horizontal chain will help much to improve the resistance of the vault.

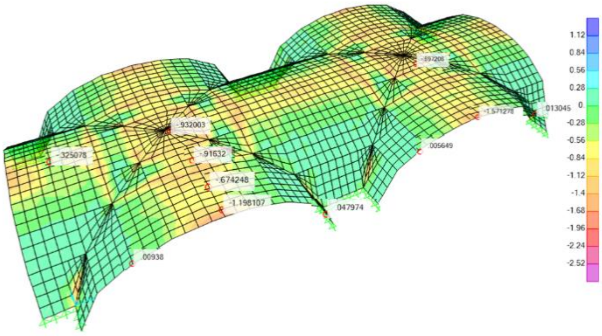
**Figure 25.** Tensile stress in the cross vault extrados

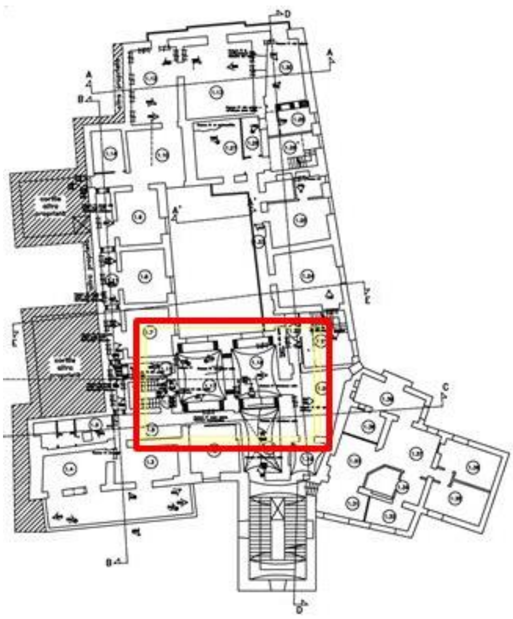
**Figure 26.** First-floor plan. In the red frame the two sail vaults.

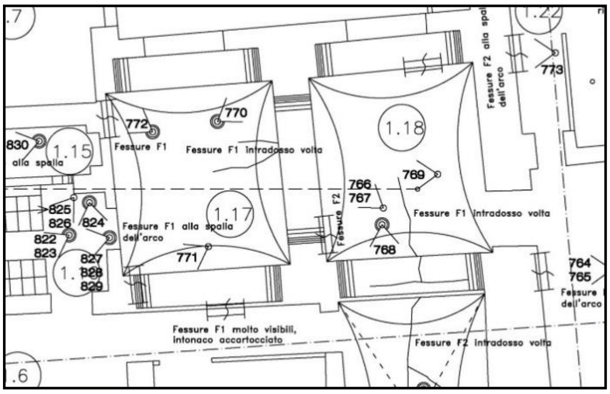
**Figure 27.** Plan of the sail vaults at the first floor


**Figure 28.** Photo of the sail vault on the first floor

We have two beautiful and fine-painted sail vaults on the first floor. The structure is similar to a dome structure, and it can be divided into a series of meridians and parallels, then this three-dimensional structure can be meshed for later finite element non-linear analysis, schematized as a grid formed by meridians and parallels. The material is assumed as for the cross vault. The dimension in the plan of the two sail vaults is about 43 square meters each vault.

For this structure has been prepared two main analyses. One is the vertical push-over analysis considering only the existing geometry. Another one has been one model considering the hypothesis of the insertion of a new chain bar.

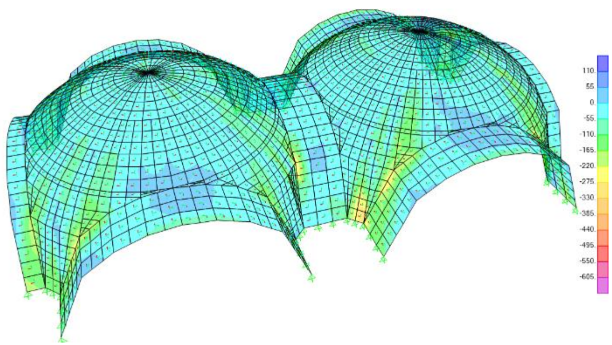
**Figure 29.** Mesh of the double sail vault and stress for ultimate live load

The numerical analysis gives a structural safety factor of the existing sail vault equal to:

η = 1.078 > 1

The value apparently is higher than 1, but we should consider the variability of several parameters, which could suddenly bring down the safety factor, and lead to the collapse. Then we decided to reinforce this double sail vault room with some chain bars which will increase safety. Two steel chain bars D=24mm have been foreseen for the common arch in the center of the room In this case the new safety factor becomes:

$$\eta = 2.1 > 1$$

which seems a satisfactory result. In “Fig. 33” the different curves of the limit analysis (pushover) are shown for the sail vault. The two chain bars will be installed at the extrados of the sail vault, as shown in “Fig. 31”, to minimize the visual impact.

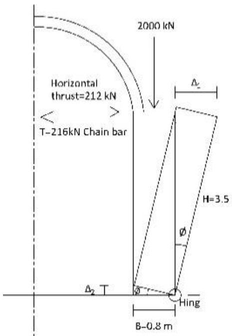


**Figure 30.** Equilibrium analysis for column, thrust, and steel chain bar for the sail vault.

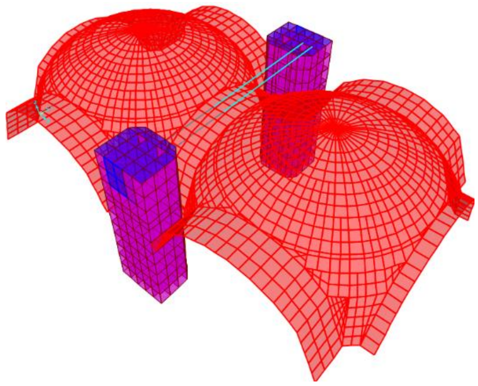

**Figure 31.** Graphic scheme of the two chain bars to be installed on the sail vaults.

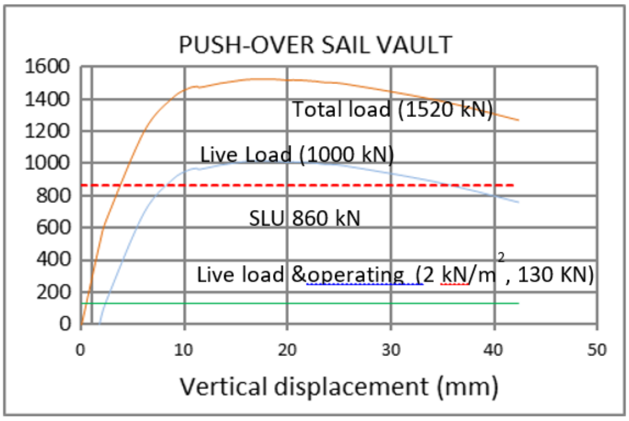


**Figure 32**. Sail vault Push-Over curves for different load conditions, without chain bar.

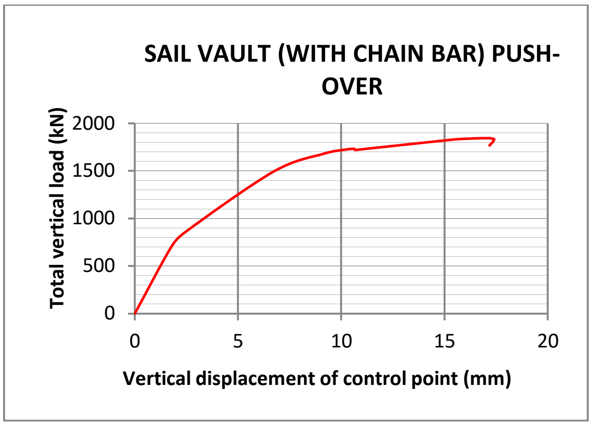


**Figure 33.** Pushover of the sail vault with the two chain bar. Total collapse load Qu= 1800 KN

As it can be seen in “Fig. 32”, the sail vault could collapse at a total vertical load of 1520 kN, above the requested 1000 kN. The added chain bar increases the ultimate load up to 1800 kN, as shown in “Fig. 33”.

The traditional staircase we can see in this building is a special one made only by masonry, in the shape of the half-barrel vault, in the transversal cross-section, and like a very shallow arch in the longitudinal direction “Fig.34”. The structural analysis of this kind of shape is still a big debate between the historical masonry structure experts. We did, as a modern engineer, a FEM model of the masonry non-linear structure. In fact, a limit analysis has been carried out considering the collapse load and evaluating a safety factor towards the operating load “Fig.37”.


**Figure 34.** Roman-style rampant staircase and dimensions

The size and the characteristics of the single analyzed element are as follows:

L=5,6 m length
B=1,9 m width
f= 0,20 m rise
S= 0,25 cm medium thickness
Material: as described before.

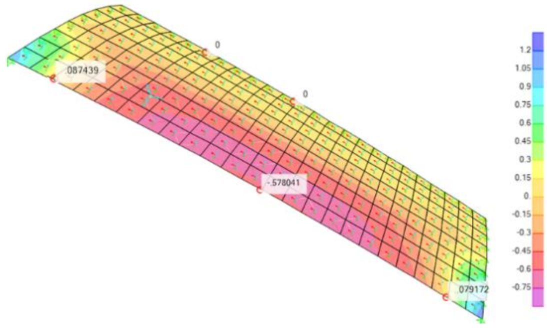

**Figure 35.** Stresses on the extrados of the stair vault due to ultimate load

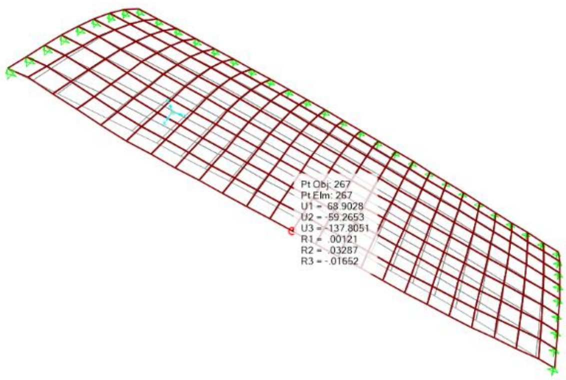

**Figure 36.** Displacement by ultimate load

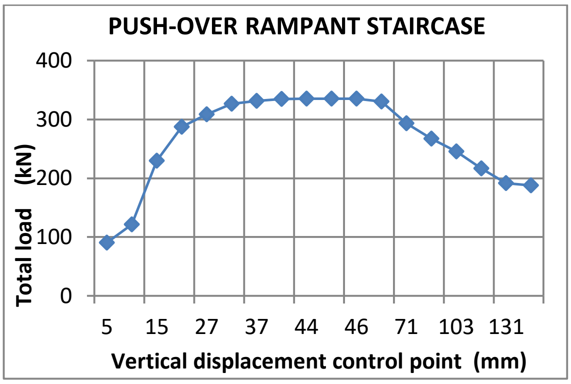


**Figure 37.** Pushover function for the load on the rampant staircase

The numerical analysis gives a total ultimate load of 335 kN, while the operating load (dead+live) is 128 kN. The ratio between ultimate live and operating live load is about

$$\eta=6$$

Since by code we need at least a safety factor of 1.9, it means that the staircase is safe enough, and doesn't need any reinforcement operation.

Finally, we check the stability of the main façade, which means we evaluate the overturning risk of the whole wall in case of an earthquake. We can consider the overturning mechanism from the ground floor to the fifth floor, for a Htotal = 23.25m. In the plan below you can see that the floor is supported by lateral walls, while the façade support itself. Then we consider the wall disconnected from the floor.

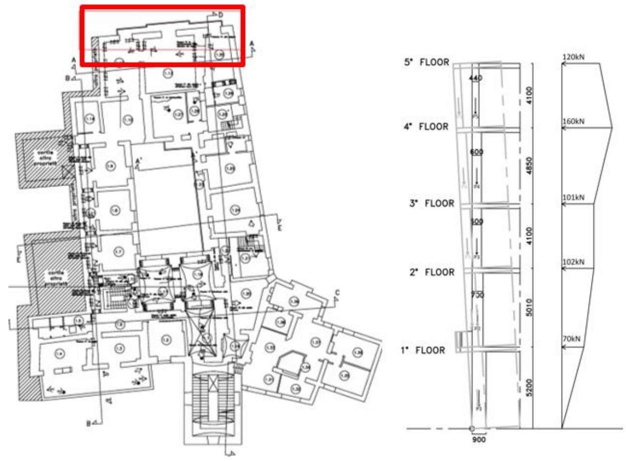

**Figure 38.** Overturning risk assessment of the main façade

In the static analysis for horizontal loads we consider the application of a system of forces distributed along with the height of the building assuming a typical linear distribution of displacements. According to the calculation, we can have:

W total = 3900 kN; Fh requested = 552 kN

$\lambda$ collapse = 0.348; F collapse = 0.348*3900kN=1357 kN

It means that with these numbers we have a presumed safety of:

$$\eta_{overturning} = F\ collapse/Fh\ requested = 2.45$$

So the wall overturns when the acceleration of gravity is greater than 0.348g.

In order to improve the masonry wall behavior, and reduce the overturning risk, we decide to put 2 pairs of chain bars on each cross wall. We foresee using a chain bar with D=24mm. The balance between internal and external work for virtual displacement, after considering also the contribute from the added chain bars at each floor, gives an interesting 30% increase of resistance for the overturning.

## 3. RESULTS

### 3.1. Global Fem Model of the Building

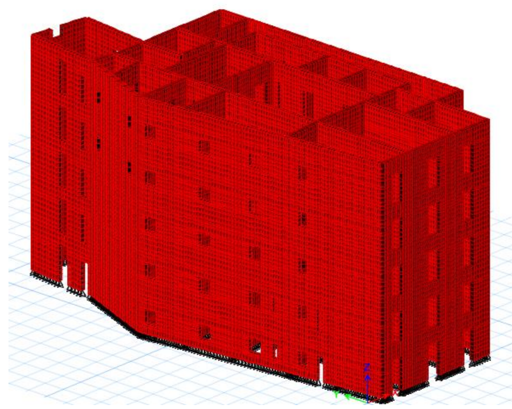

**Figure 39.** Global FEM model for the non-linear analysis of the building

After checking and verifying a single structural element, and improving the local static and seismic resistance, a finite element model of the global structure, it means the total building, has been carried out utilizing the well-known code ETABS. The non-global linear analysis (pushover) allows to follow the decay process of the structure with a monotonic load increasing. In this way it is also possible to optimize the reinforcement process, installing the chain bars and other reinforcement in the most suitable location.

The first vibration period for the model of the building in the existing condition is about 0.65 sec, while after the reinforcement with the chain bars the first vibration period reduced down to about 0.35 sec.

**Table 2.** Modal analysis results

| Case | Mode | Period | Frequency | Circular Frequency | Eigenvalue |
|---|---|---|---|---|---|
| | | sec | cyc/sec | rad/sec | $rad^2/sec^2$ |
| Modal | 1 | 0,337 | 2,965 | 18,6286 | 347,0246 |
| Modal | 2 | 0,26 | 3,843 | 24,1483 | 583,1386 |
| Modal | 3 | 0,24 | 4,169 | 26,1955 | 686,2046 |
| Modal | 4 | 0,222 | 4,496 | 28,2523 | 798,1911 |
| Modal | 5 | 0,219 | 4,568 | 28,6991 | 823,6385 |
| Modal | 1 | 0,337 | 2,965 | 18,6286 | 347,0246 |

**Table 3.** Modal Mass Participation ratio

| Case | Mode | Period sec | UX | UY | Sum UX | Sum UY | RZ | Sum RX | Sum RY | Sum RZ |
|---|---|---|---|---|---|---|---|---|---|---|
| Modal | 1 | 0,337 | 0,7867 | 0,0014 | 0,7867 | 0,0014 | 0,0027 | 0,0003 | 0,3407 | 0,0027 |
| Modal | 2 | 0,26 | 0,0023 | 0,0032 | 0,789 | 0,0046 | 0,7152 | 0,0008 | 0,3408 | 0,7179 |
| Modal | 3 | 0,24 | 0,0004 | 0,7036 | 0,7894 | 0,7082 | 0,0047 | 0,202 | 0,3415 | 0,7226 |
| Modal | 4 | 0,222 | 0,001 | 0,0197 | 0,7903 | 0,7279 | 0,0064 | 0,2258 | 0,3416 | 0,729 |
| Modal | 5 | 0,219 | 0,0035 | 0,0727 | 0,7938 | 0,8005 | 0,0115 | 0,2772 | 0,3416 | 0,7405 |

The comment to these results is that the good quality of the reinforcement is proved by the increase of global stiffness of the building (lower vibration period) and by the high modal mass participation ratio achieved after the first five vibration modes (about 80% of total mass).

### 3.1. Details of the Reinforcing Design

At this point the main work from the structural designer becomes very crucial, it means to decide where and how to install the reinforcing devices.

If the building has some historical value, and shows some decoration, internally and externally, the designer has to preserve the architectural cultural heritage and try to use the less invading techniques and technology for this purpose.

**Figure 40.** Rendering of some of the structural reinforcing on the external wall of the building

Our choices were related to ("Fig. 40"):

- use of extrados chain bar in the sail vault at the first floor, reducing the visual impact on the existing room;
- use of double (twin) stainless chain bar for each wall that needs to be linked to the other wall, drilling small holes in the wall, avoiding vibrations;
- using vertical confining stainless metallic profiles to better protect the corner of the masonry at every level, and not only at floor level;
- use of prestressed horizontal carbon fiber stripes, for active confinement of the wall.

The use of stainless steel is a good way to perform a durable reinforcing action, without fear of seeing the chain bars or the steel profiles becoming rusty.

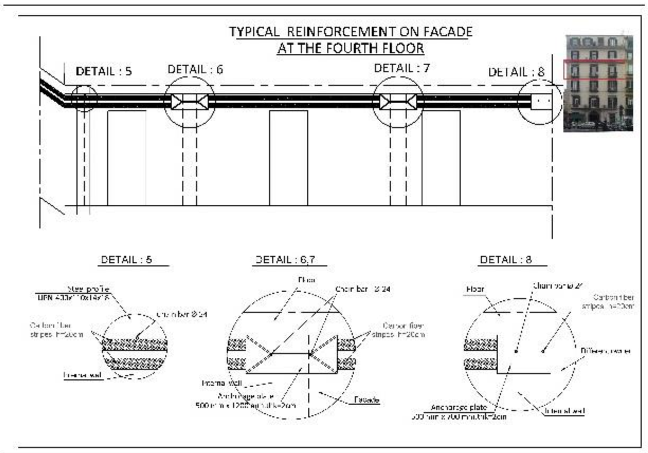


**Figure 41.** Details for the installation of the prestressed carbon fiber and the plates for the anchorage of the stainless chain bar.

The use of drilling in the masonry walls without adding percussion and vibration to the drilling is a good way to avoid additional stress to the old masonry elements.

Finally, the use of carbon fiber stripes is another choice in the direction of the "reversibility" of the intervention, since the small quantity of resin adopted to bond the fiber to the masonry can be easily removed, together with the fiber. But also the steel bar and all the other proposed interventions will use very limited concrete, which could be very difficult to remove in case of need. In "Fig. 41".

It is possible to see the installation details for what described

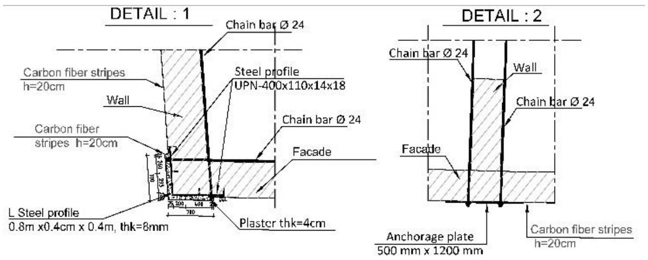


**Figure 42.** Detail of carbon fiber stripes and chain bar

**Figure 43.** Anchorage in the masonry with the injection of grout limited by a sock.

## 4. CONCLUSIONS

The paper shows that even residential buildings with some historical value due to the age and also to the construction typology deserve to be preserved for their cultural and architectural heritage value.

The introduced case is related to a residential building in one area in Naples of high historical architecture value. The building itself is more than 200 years old and still with the remaining sign of its history. Nevertheless, the building during the centuries has been involved in several changes because the several owners had different needs about their home, and today, due to also to some deep local structural change, some risk can be considered for seismic events and even for daily static behavior. The technology today allows working more softly in reinforcement in order to give back to the masonry structure a correct structural safety. Technology also allows preserving the authenticity and the architectural heritage. In this professional work, the design team considered very carefully the use of advanced material like carbon fiber stripe but also stainless steel chain bar to obey the principles of the material compatibility and reversibility of the intervention. Also, the use of stainless steel is a good way to perform a durable reinforcing action, without fear of the rust on the metal. After the work, all the reinforced parts will be hidden under a little layer of the plaster, and even the plaster will be carefully removed from the surfaces, and replaced with compatible material. At the end of the work, only the final steel plates for the anchorage of the chain bars will be visible from outside and even that on could be partially hidden under the existing thick plaster with its decoration.

## Author contributions

Srey Mom Vuth: Conceptualization, Original draft preparation, Methodology,
Paola Paterna: Data curation, Extended analysis, Validation.
Donato Abruzzese: Visualization, Investigation, Writing-Reviewing and Editing.

## Conflicts of interest

There is no conflict of interest between the authors.

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**Cultural Heritage and Science**
https://dergipark.org.tr/en/pub/cuhes
*e-ISSN 2757-9050*

# Study on the Influence of Knots on the Seismic Performance of Chinese Traditional Wooden Building Beams

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**Keywords**
Wooden Beam,
Wood Knots,
Seismic Performance.

**ABSTRACT**

Log is widely used in Chinese traditional wooden buildings, which has good compression, bending, tensile and seismic performance, but the existence of knots in log will have a greater adverse impact on the structure. Taking the residence of Zhou Fujiu, a salt merchant in Yangzhou as an example, the building was built in the late Qing Dynasty, covering an area of 3700 square meters. Through the field investigation of Zhou Fujiu building in Yangzhou, it is found that in addition to the common natural cracks, the wooden beams with knots in the tension zone are obviously damaged. In order to further study the influence of knots on the seismic performance of wooden beams and provide relevant theoretical basis for the protection of traditional Chinese wooden buildings, six different wooden beams were designed according to the scale of 1:5.28, and the low cycle reciprocating loading test under three-point loading was carried out. The effects of knots at different positions and depths on the seismic performance of timber beams are simulated respectively. The hysteretic curve and skeleton curve of the timber beam with knots are obtained by experiments, and the stiffness degradation and energy dissipation capacity are analyzed. The results show that with the increase of the depth of the knots, the bearing capacity of the beam shows a significant decreasing trend, the hysteretic curve becomes more stable, the energy dissipation capacity decreases, and the seismic performance decreases. When there are knots on both sides of the bending and compression area, the seismic performance of the wooden beam decreases most significantly.

## 1. INTRODUCTION

Wood structure architecture is a typical representative of Chinese ancient architecture, which has high artistic value and practical value. Systematic analysis and research on it will help us to take better protection measures. Wooden beam is a common component in ancient buildings. It often bears the load from the roof and is an important force transfer component. The mechanical performance of wooden beam is related to the overall seismic performance of the building. For different materials and different structural forms of wood beam performance, scholars have carried out more in-depth research. Corradi et al. (2021) carried out FRP reinforcement on the defective small wooden beams, and evaluated their bending performance under static load, which confirmed the effectiveness of the reinforcement scheme. Jeong-Moon et al. (1999) conducted an experimental study on a residential building with wooden beams in North Korea, and found that its load displacement curve has obvious nonlinear and inelastic characteristics. Yeboah and Gkantou (2021) proposed a general theoretical model to estimate the flexural capacity of nsm-frp wood beams. Nubissie et al. (2011) conducted a quantitative study on the fire resistance of wood beams. A combined analytical and numerical model for predicting the fire resistance of a wooden beam is presented. Gribanov et al. (2020) studied the strength of wooden beam structures with local modification of wood in the compressed zone.

Many ancient wooden buildings in China are still standing, but with the passage of time, the impact of wood damage on the overall seismic performance of buildings is becoming increasingly prominent. In recent years, research on damaged wood structures has made some progress. Irbe et al. (2005) studied the wood

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**Cite this article**

*Li Cai S, Fan T & Abruzzese D (2021). Study on the Influence of Knots on the Seismic Performance of Chinese Traditional Wooden Building Beams.* Cultural Heritage and Science, 2(2), 43-49

**Research Article**

**Received: 16/11/2021; Accepted: 22/12/2012**

quality loss under different fungal corrosion conditions. Campilho et al. (2009) made an experiment on the bending behavior of wood beams of the Pinus Pinaster species repaired with adhesively-bonded carbon–epoxy patches. In addition, Karagöz and Kesik (2021) have studied that carbon fiber-reinforced polymer (CFRP) strips were used to improve the mechanical behaviors of old wood samples that were damaged over time because of various environmental effects and biological factors. At present, the research on wood structure damage mainly focuses on decay and cracking, but the research on knots is rare. There are three main types of wood beam damage in Yangzhou Zhou Fujiu building, which are cracks in square beam (Fig. 1), cracks in circular beam (Fig. 2) and damaged beam with knots (Fig. 3). It can be seen that the timber beams with knots have significant damage in the tensile zone, which will have a negative impact on the overall seismic performance of the building. Based on the above phenomena, this paper studies the influence of knots on the seismic performance of wooden beams.

**Figure 1.** Cracks in square beam

**Figure 2.** Cracks in square beam

**Figure 3.** Damaged beam with knots

## 2. METHOD

In the process of field investigation of Zhou Fujiu building timber beam in Yangzhou, it is found that when there are knots in the mid span bending area of the beam, the damage effect is the most significant. Based on this, the method of slotting in the middle of the wooden beam span is used for artificial simulation, and the simulated knots have the following characteristics: the beam section decreases when in tension; When pressed, it shows mutual extrusion and sliding. The height to width ratio of wood beam section is 3:2. Combined with the actual size of the wood beam of Zhou Fujiu building in Yangzhou, the scale of the model is 1:5.28. A total of 6 wood beams with a width of 80mm, a height of 120mm, a span of 2300mm and a clear span of 2160mm are manufactured. The wood variety is American Douglas fir.

The slot position is set in the middle of the span, and two slots with a width of 2mm are set. The Members S1-1, S1-2 and S1-3 simulate the existence of knots in the bending area of timber beams (Fig. 4); The members s2-1, s2-2 and s2-3 simulate the situation that there are knots on both sides of the bending zone and tensile zone (Fig. 5).

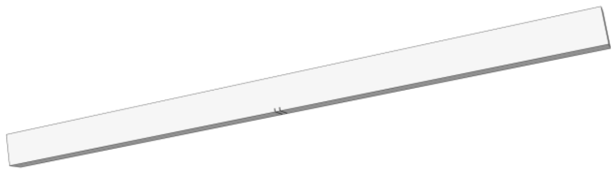


**Figure 4.** Three-dimensional schematic diagram of S1-1, S1-2 and

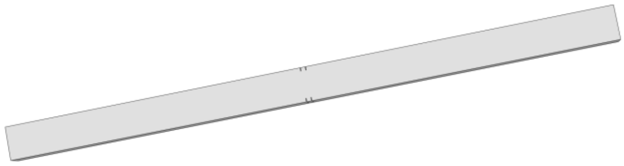


**Figure 5.** Three-dimensional schematic diagram of S2-1, S2-2 and S2-3

The depth of the groove is designed as 1 / 10, 1 / 5 and 1 / 3 of the beam height, 12mm, 24mm and 40mm respectively (Table 1, 2).

**Table 1.** Schematic diagram of S1

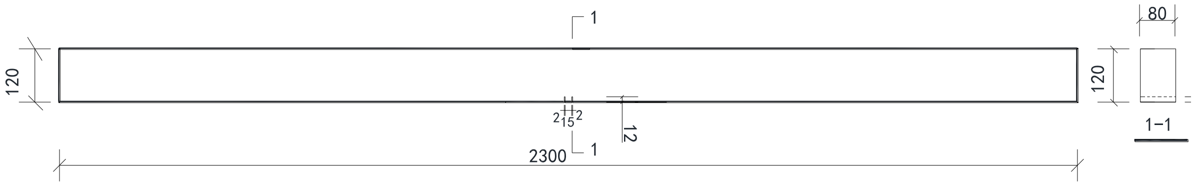
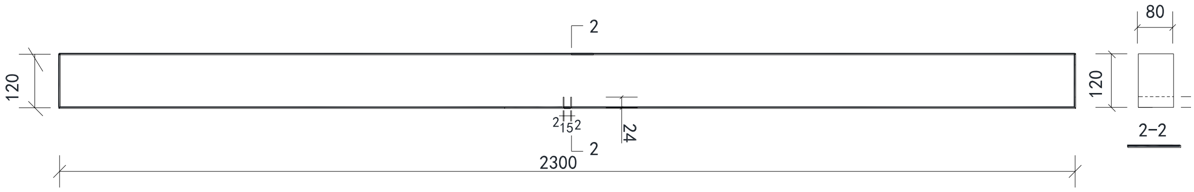
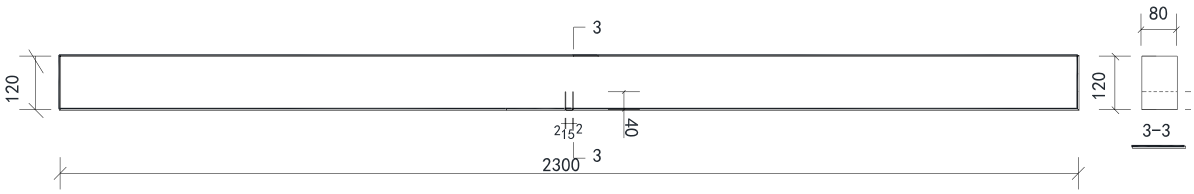
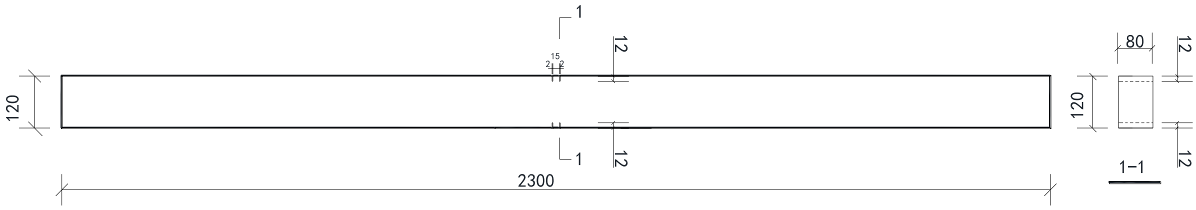
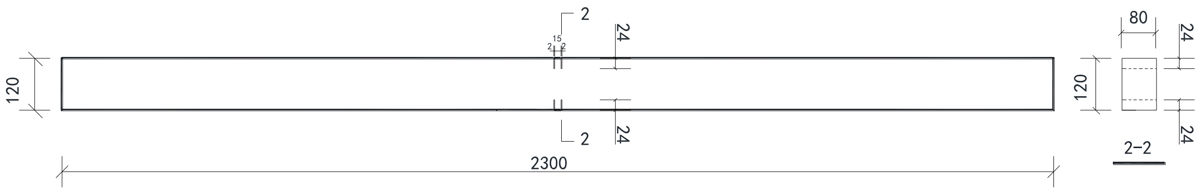
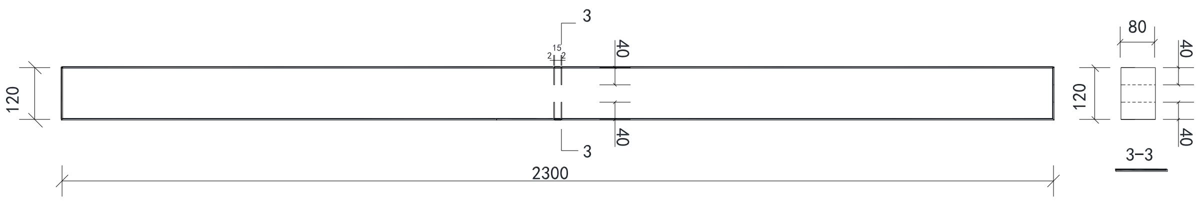
| Number | Diagram |
|---|---|
| S1-1 |  |
| S1-2 |  |
| S1-3 |  |

**Table 2.** Schematic diagram of S2

| Number | Diagram |
|---|---|
| S2-1 |  |
| S2-2 |  |
| S2-3 |  |

## 2.1. Loading Scheme

The specimen is loaded at three points, and the distance between the two loading points is 6 times of the section height, i.e. 720mm (Fig. 6).

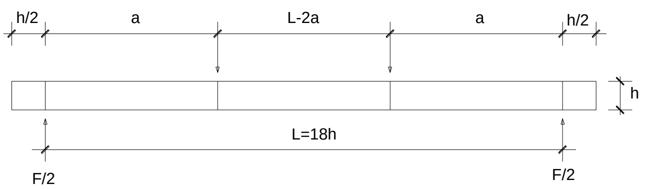



**Figure 6.** Schematic diagram of three point loading test of wooden beam

The low cycle reciprocating force is applied by the servo loading hydraulic press and the mode of displacement control loading is selected. The amplitude of displacement change is 10 mm during loading, and three cycles are carried out for each stage (Fig. 7).

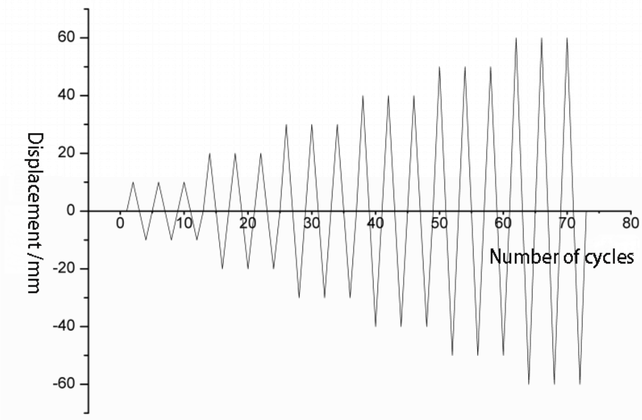



**Figure 7.** Loading mode of wooden beam

### 2.2. Experimental Phenomena

The damage of the six components finally occurred at the knots, which is consistent with the actual damage of the wooden beams of Zhou Fujiu building in Yangzhou. The experimental phenomena of the three members in group S1 are basically the same, and the cracks appear first on the side where the knots exist, and the deeper the knots are, the earlier the cracks appear. With the progress of loading, the cracks on one side of the knot gradually expand and eventually develop to the whole mid span section, resulting in the failure of the member (Fig. 8). There are knots on both sides of the middle span of the three members in group S2, and obvious cracks appear at the initial stage of loading. With the increase of displacement value, inclined cracks gradually appear along the middle of the span to both sides, and the width and length of cracks increase continuously until the members are completely destroyed (Fig. 9).

**Figure 8.** Midspan failure of S1 group

**Figure 9.** Midspan failure of S2 group

## 3. RESULTS

The hysteretic curves of S1 group and S2 group are obtained through the test, and the curves are processed and analyzed to study their seismic performance.

### 3.1. Hysteresis curve

By comparing and analyzing the hysteretic curves of six wooden beams (Fig. 10, 11, 12, 13, 14, 15), we can draw the following conclusions: 1) S1 group wooden beams only have knots on one side of the middle span, the hysteretic curves show obvious asymmetry, and the bearing capacity of the side with knots is lower than that of the intact side in each stage of loading. There are simulated knots on both sides of the middle span of S2 group timber beam, so the hysteretic curve is basically symmetrical. 2) At the later stage of loading, the stress concentration will appear at the knots, the bearing capacity of the beam will decrease rapidly, and the hysteretic curve will become an inverse S-shape. 3) When other conditions are the same, with the increase of node depth, the bearing capacity decreases more. S1-3 and S2-3 of S1 group and S2 group are the members whose bearing capacity decreases most. 4) When the depth of the knots is the same, the existence of knots on both sides is the most unfavorable to the bearing capacity of the beam.

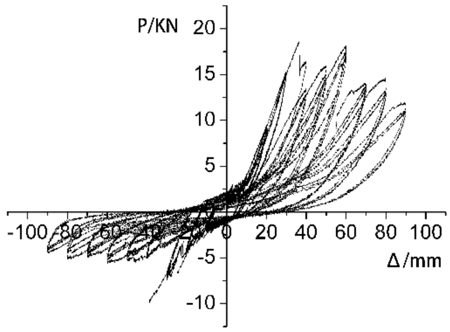



**Figure 10.** S1-1 hysteresis curve

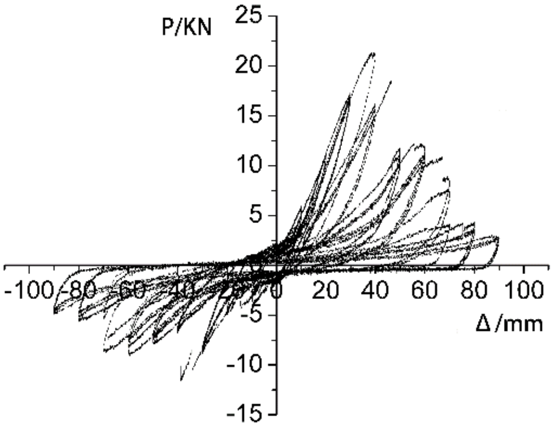



**Figure 11.** S1-2 hysteresis curve

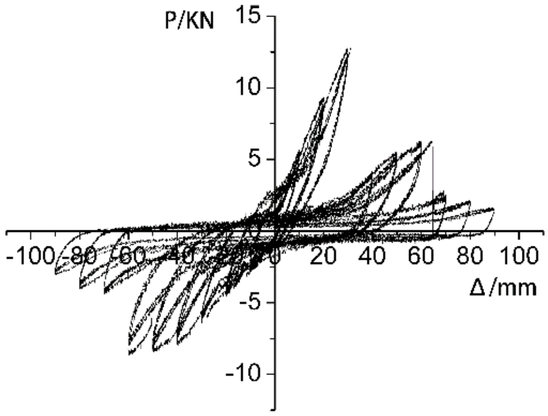



**Figure 12.** S1-3 hysteresis curve

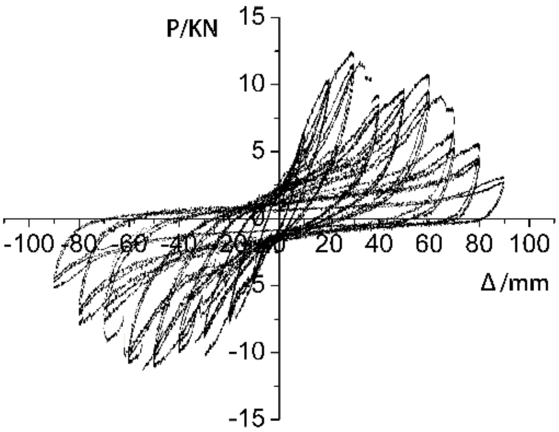



**Figure 13.** S2-1 hysteresis curve

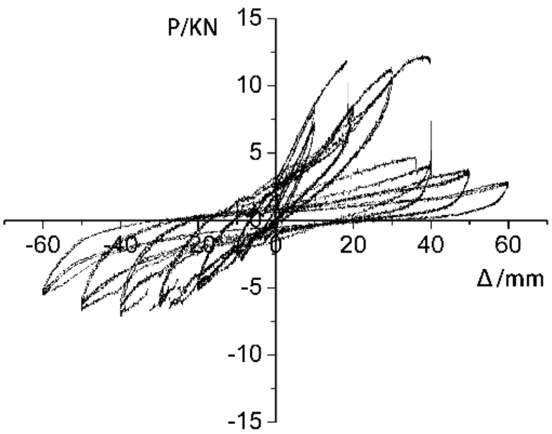



**Figure 14.** S2-2 hysteresis curve

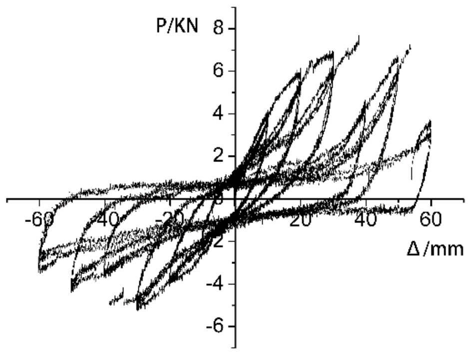



**Figure 15.** S2-3 hysteresis curve

### 3.2. Skeleton Curve

By analyzing the skeleton curves of S1 and S2 groups (Fig. 16, 17), it can be found that the deeper the knot depth is, the earlier the failure time is. Taking group S1 as an example, when the displacement is 30mm, the S1-3 member will enter the failure stage, which is obviously earlier than the other two members in the same group.

In the process of forward loading, the ultimate load of S1-1 member with 12 mm knot depth is 17.2 KN, while that of S1-3 member with 40 mm knot depth is 12.6 KN, which is 26.7% lower than that of S1-1. The decrease of ultimate load is obviously related to the increase in knot depth. Comparing the skeleton curves of the two groups of members, it can be found that when there are knots on both sides of the bending zone and tensile zone, the bearing capacity of the wooden beam decreases more significantly. Under positive loading, the depth of the knots of S2-1 is the same as that of S1-1, both are 12mm, but the ultimate load is only 12.7 KN, with a decrease of 26.1%.

Among the six members, S2-3 is the most disadvantageous in the depth and distribution of knots. There are knots with a depth of 40mm on both sides of the bending zone and tensile zone of the member, and the bearing capacity decreases most. In the forward loading process, the ultimate load of S2-3 is only 7.6 KN, which is 55.8% lower than that of S1-1 member with the least damage.

Through the above data analysis, it can be found that the influence of knots on the mechanical properties of wooden beams can not be ignored, and the depth and location of knots are important factors. If there are knots in the actual wooden beams, the possible adverse effects should be evaluated in time, and effective reinforcement or replacement measures should be taken.

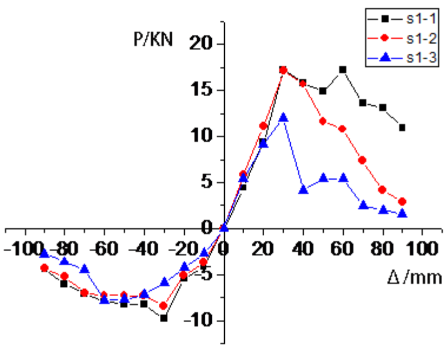



**Figure 16.** Skeleton curve of S1

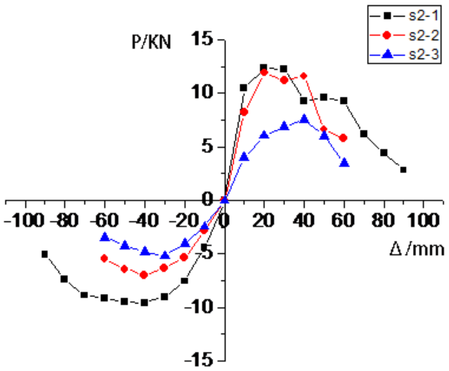



**Figure 17.** Skeleton curve of S2

### 3.3. Stiffness Degradation

It can be seen from the degradation curves that the stiffness of the six members is degraded in the process of low cycle reciprocating loading (Fig. 18, 19). The initial stiffness of the members is larger, but with the increase of loading displacement, the cracks of the members develop along the knots, leading to the gradual decline of stiffness. The degradation range of member stiffness increases with the increase of knot depth. In group S1, the stiffness degradation of S1-3 is the fastest; In S2 group, S2-3 members enter the plastic stage quickly, and the stiffness decreases significantly at the initial stage of loading.

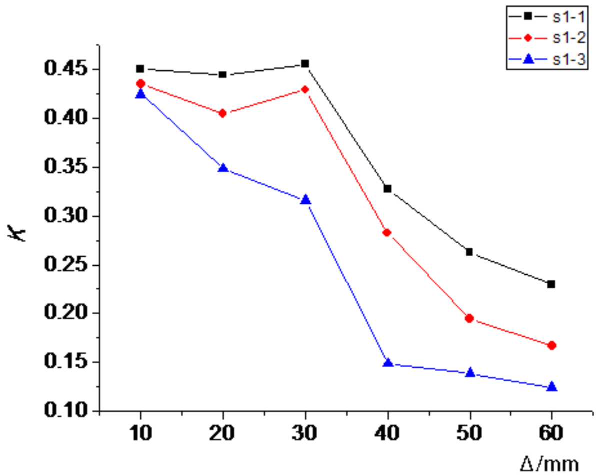



**Figure 18.** Stiffness degradation curve of S1

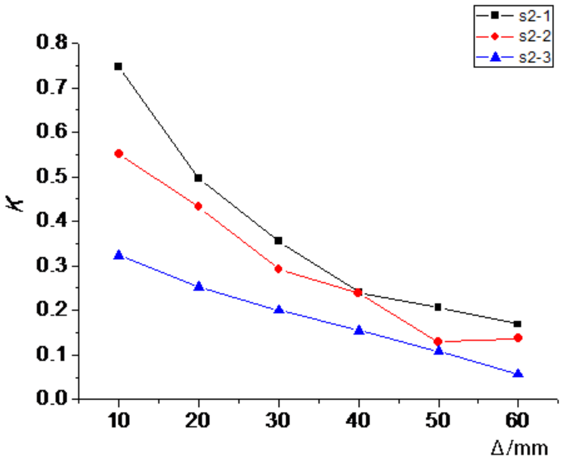



**Figure 19.** Stiffness degradation curve of S2

### 3.4. Energy Consumption Capacity

With the increase of displacement, the energy dissipation coefficient increases slightly in the initial stage and decreases rapidly in the later stage. The energy dissipation capacity of members with different depth and position of knots is different. The energy consumption coefficient of S1 group reaches temporary stability when the displacement value is 20 mm, and increases slightly (Fig. 20); When the displacement value is 40 mm, the energy dissipation coefficient decreases rapidly, and the member is damaged. It can be seen from the figure that S1-1 with the shallowest knot depth has the best energy dissipation capacity. It can be found that when there are wooden joints in the bending area and tensile area of the wooden beam, the energy consumption coefficient of the member is lower, and the decline section of the energy consumption coefficient appears earlier. When the displacement value is 30mm, the energy consumption force begins to decrease significantly (Fig. 21).

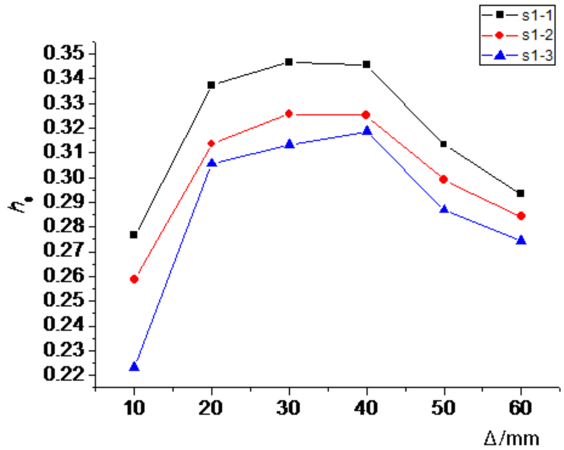



**Figure 20.** Energy consumption curve of S1

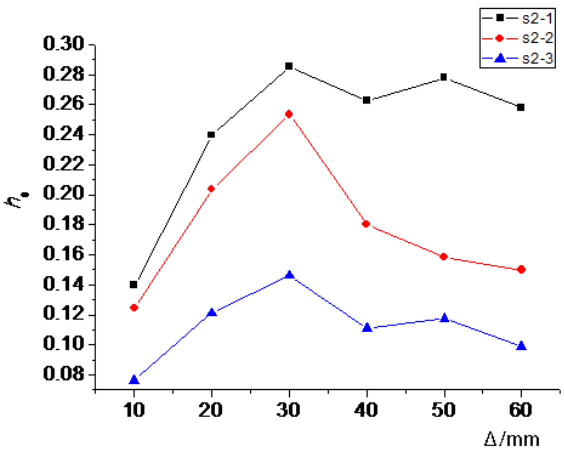



**Figure 21.** Energy consumption curve of S2

## 4. CONCLUSION

1) In the test, the failure position of the wooden beam mainly occurs in the middle of the span, and the failure cracks develop from the middle of the span to both sides of the beam. The failure mode is consistent with the actual failure mode of Zhou Fujiu building in the investigation site.

2) The deeper the depth of the knot, the weaker the bearing capacity, the more significant the stiffness degradation, the weaker the energy dissipation capacity and the worse the seismic performance. Especially when the depth of the knot reaches 1 / 3 of the beam height, the beam is not suitable for continuous bearing, which should be paid enough attention to.

3) When there are knots in the mid span bending area and tension area, the bearing capacity of the timber beam will lose rapidly, which will affect the stability of the whole building structure, which is the most unfavorable to the earthquake resistance.

4) In the maintenance of traditional Chinese wooden buildings, knots should be strictly monitored and their harmfulness should be evaluated in time. When there are deep knots in the middle of the wooden beam span, measures such as rigid wrapping should be taken for reinforcement. When there are knots in the bending area and tensile area of the wooden beam span and the depth is deep, they should be replaced in time.

**Acknowledgments**

The work has been supported by International cooperation research program of Yangzhou (Grant No. YZ2019148), the National Natural Science Foundation of China (Grant No. 51478409), the State Administration of Foreign Experts Affair of China (Grant No. G2021014061L), and innovative sci-tech platform cooperation project of between Yangzhou City and Yangzhou University (Grant No. YZ2020262).

**Author contributions**

Shengcai Li: Investigation, Conceptualization, Methodology, Writing-Original draft
Tianhao Fan: Data curation, Writing-Original draft preparation, Software, Validation.
Donato Abruzzese: Validation, Writing-Reviewing and Editing.

**Conflicts of interest**

The authors declare no conflicts of interest

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Cultural Heritage and Science
https://dergipark.org.tr/en/pub/cuhes
e-ISSN 2757-9050

# Seismic Assessment of Four Historical Masonry Towers in Southern Italy

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**Keywords**
Masonry towers,
On-site surveys,
Dynamic testing,
Seismic assessment,
Pushover analysis,
Safety index.

**ABSTRACT**

The paper synthesizes the main results of investigations devoted to the evaluation of the seismic performance of four historical masonry towers. On-site diagnostic investigations were carried out using non-destructive or slightly destructive tests, including geometric surveys, laser scanning, endoscopic tests, sonic pulse velocity tests, geognostic surveys, flat-jack tests, environmental vibration tests, dynamic tests on tie-rods. The results of the on-site surveys were employed to calibrate refined 3D finite element models of the towers accounting for the materials' mechanical parameters, restraint of the neighboring constructions, and effect of soil-structure interaction. The FEM model was usefully employed to assess the seismic risk of the towers based on the Italian Guidelines. To this aim, the nonlinear FEM global analysis was developed using the pushover technique for the estimation of the seismic safety of the towers.

## 1. INTRODUCTION

Many ancient masonry structures are vulnerable to lateral inertial forces due to seismic events. The poor ductility of the masonry, combined with other elements of weakness such as irregular geometry or slenderness, may increase the risk of severe damages or collapse even under moderate earthquake ground motions. The lessons learned from earthquakes in recent decades indicate the vulnerability of the historical constructions and the need to ensure the seismic safety of cultural heritage. This has given rise to a new generation of international guidelines and codes [Eurocode 8, 2004; FEMA 356, 2000; ASCE/SEI 31-03, 2003; FEMA 547, 2006; ASCE/SEI 41-06, 2007; DPCM, 2011; ASCE/SEI 41-13, 2014] that include specific procedures to evaluate the seismic safety of historical constructions. Moreover, the present circumstance stimulated the advancement in models and analysis methods for this type of structure. The tall monumental buildings, for example, towers and mosque minarets, don't have sufficient structural capacity to resist to seismic actions. Some of the major deficiencies relate to geotechnical problems, intrinsic structural defects, degradation of materials, buckling of slender members, and sensitivity to dynamic loading. Many studies in the literature focused on the damage evaluation and seismic assessment of historical constructions, with particular concern to ancient masonry towers (Lagomarsino &. Cattari, 2015; Valente & Milani, 2016; Fragonara *et al.*, 2017; Bartoli *et al.*, 2017). A large number of studies developed ambient vibration tests (AVTs) on old buildings and suitable experimental techniques for their dynamic identification (De Sortis *et al.*, 2005; Gentile *et al.*, 2015; Ferraioli *et al.*, 2018). Bayraktar *et al.* (2009) presented a study on the dynamic identification and model updating of the Hagia Sophia bell tower in Turkey. Russo *et al.* (2010) performed an experimental campaign aimed at the evaluation of the performances of the bell tower of Saint Andrea in Venice. Tomaszewska *et al.* (2012) presented the dynamic identification of the Vistula Mounting tower. D'Ambrisi *et al.* (2012) used the ambient vibration tests to identify first the dynamic modal parameters and then the material properties of the civic tower of Soncino in Cremona (Italy). Gentile *et al.* (2015) proposed a vibration-based procedure to calibrate the finite element model of the bell tower of the Church Collegiata in Arcisate (Varese, Italy). Preciado (2015) focused on towers and other slender masonry structures developing a seismic vulnerability assessment method. Casolo *et al.*

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**Cite this article**

*Ferraioli M & Abruzzese D (2021). Seismic Assessment of Four Historical Masonry Towers in Southern Italy*. Cultural Heritage and Science, 2(2), 50-60

**Research Article**

**Received: 09/11/2021; Accepted: 08/12/2021**

(2013) carried out a comprehensive study where ten masonry towers in the coastal Po Valley (Italy) were analyzed, and their seismic vulnerability was evaluated and discussed. On the other side, many studies focused on the mechanical model of masonry that should be able to characterize all the aspects of its complex non-linear behavior (Bernardeschi *et al.*, 2004; Carpinteri *et al.*, 2005; Peña *et al.*, 2010, Milani *et al.*, 2012). This paper synthetically reports the experience of the extended experimental and theoretical campaign dedicated to four historical masonry towers in Southern Italy. The experimental campaign included geometric surveys, laser scanning, endoscopic tests, flat-jack tests, sonic pulse velocity tests, environmental vibration tests, geognostic surveys, and dynamic tests on the tie-rods. The ultimate goal of these activities was to address several uncertainties related to the geometry and mechanical properties of materials that, along with soil stratigraphy and site seismicity, address a critical part of the seismic evaluation of these structures. The on-site investigation survey was the basis to calibrate refined nonlinear finite element models that were employed to evaluate the seismic safety of the towers using the capacity spectrum method (Fajfar, 1999) based on the inelastic demand spectra.

## 2. INVESTIGATIONS, ON-SITE SURVEYS, and TESTS

This paper presents the results of a comprehensive study carried out on four historical masonry towers in Campania, Italy (Fig. 1): 1) the bell tower of the Cathedral of Aversa; 2) the bell tower of the Cathedral of Capua; 3) the bell tower of the Church of Assumption in Santa Maria a Vico; 4) the bell tower of Torre Orsaia (Cilento National Park in Southern Italy).

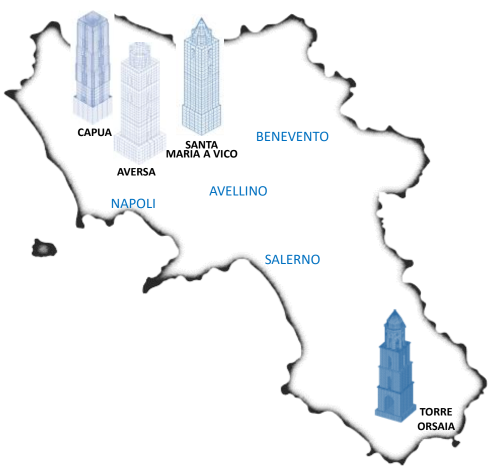



**Figure 1.** The geographical location of the towers

### 2.1. The Bell Tower of Aversa

The bell tower of the Cathedral of Aversa (Fig. 2), originally built between 1053 and 1080 next to the Lombard church, was rebuilt in 1499 after the earthquake of 1457. The tower is 45.5 m tall and its cross-section is a square of side 14 m. Four massive masonry piers on the edge of the square cross-section are coupled by spandrels with masonry arch above the openings. The horizontal structures are composed of masonry vaults for the first floor, and wooden structures for the upper floors. However, they have low in-plane stiffness and, therefore, they are not able to guarantee an adequate strong constraint linking the four masonry piers. The tower has been the subject of investigations, on-site surveys, and tests including geometrical material and survey of the crack pattern, chemical tests, single flat-jack tests, and monotonic compressive tests (Fig. 3). Finally, full-scale ambient vibration tests (Fig. 4) were carried out allowing the identification of the modal parameters of the tower (Ferraioli *et al.*, 2017). The foundations have not been the subject of any investigation. The underlying soil is bedrock made of compact Campanian grey tuff.

### 2.2. The Bell Tower of Capua

The bell tower of the Cathedral of Capua (Fig. 5), originally built in 856, was rebuilt in the 12th century after the earthquake of 990. The tower is 41 m tall and its cross-section is a square of side 11.3 m. The first two levels are composed of limestone blocks removed from antique constructions, while the upper part of the structure is composed of clay bricks and Campanian tuff blocks. The soil under the construction is composed mainly of pyroclastic sedimentary rocks that are typical of the volcanic Phlegrean area. The ambient vibration tests (Fig. 6) were carried out to measure the modal parameters (i.e. mode shapes and natural frequencies). This information was usefully applied to identify all the unknown parameters of the finite element model.

### 2.3. The Bell Tower of Santa Maria a Vico;

The bell tower of the Church of Assumption in Santa Maria a Vico (Fig. 7), originally built during the 15th century, was retrofitted in 1749 after the earthquake of 1732. The tower is 40 m tall and its cross-section is a square of side 8.3 m. The vertical load-bearing structures are made of tuff masonry with a variable thickness along with the height. The horizontal structures are masonry vaults for the first three levels and a wooden floor strengthened with a concrete slab for the fourth level. Horizontal tie bars are inserted in both horizontal directions at 27, 30 m, and 33 m from the ground level. The soil stratigraphy is composed of two layers. The first layer is made of pyroclastic loose rocks. The second layer is made of Campanian ignimbrite. The modal parameters were identified using ambient vibration tests under traffic and wind loading (Fig. 8) (Ferraioli *et al.*, 2018).

### 2.4. The Bell Tower of Torre Orsaia

The bell tower of the church of San Lorenzo Martire in Torre Orsaia is located in the Cilento National Park (Southern Italy). The building was originally a three-story fortress in the Lombard age. In 1576 the overall height was increased to about 30 m by adding two more orders with an octagonal plan and a conical roof (Fig. 9). The vertical load-bearing structures are made of local sandstone. The structure of the octagonal belfry has an internal structure made of local bricks and stones covered with a cement-based plaster.

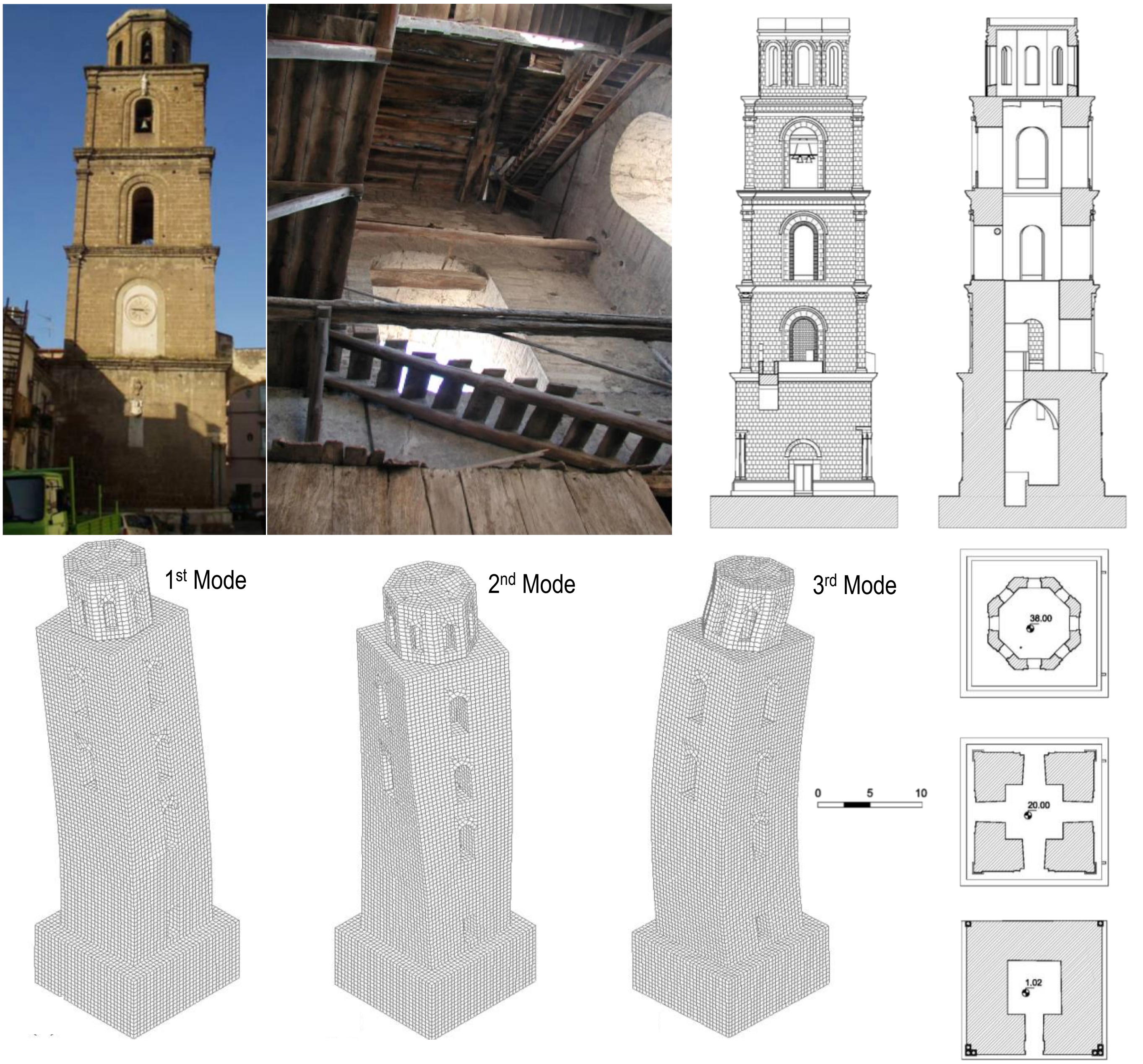
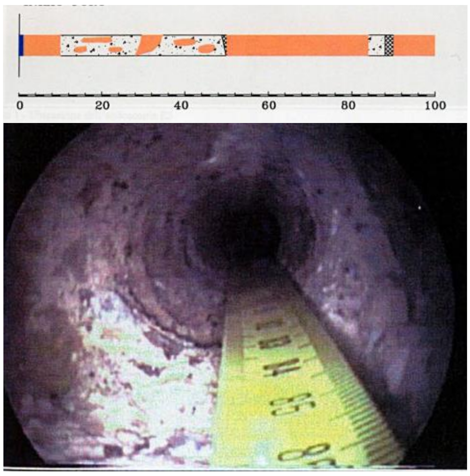
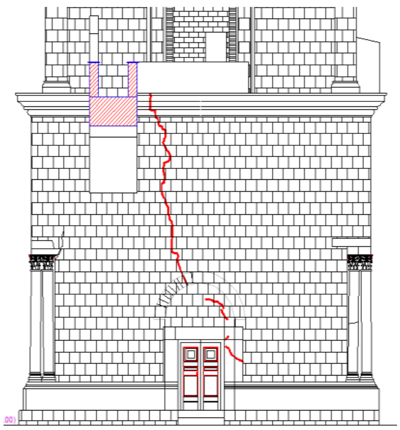



**Figure 2.** The bell tower of the Cathedral of Aversa: general view, internal view, façade, sections, mode shapes




*Endoscopic test*



*Survey of the crack patterns*

| Specimen | 1 | 2 | 3 |
|---|---|---|---|
| Description | yellow tuff with grey pumices | yellow tuff (lithoid facies) | yellow tuff with grey pumices |
| Compressive Strength [MPa] | 4.23 | 2.85 | 3.46 |
| Tangent Modulus of Elasticity [GPa] | 3.99 | 3.66 | 1.92 |
| Secant Modulus of Elasticity [GPa] | 3.82 | 3.33 | 2.08 |

*Flat-jack tests*

**Figure 3.** The bell tower of the Cathedral of Aversa: endoscopic tests, surveys of the crack pattern, flat-jack tests

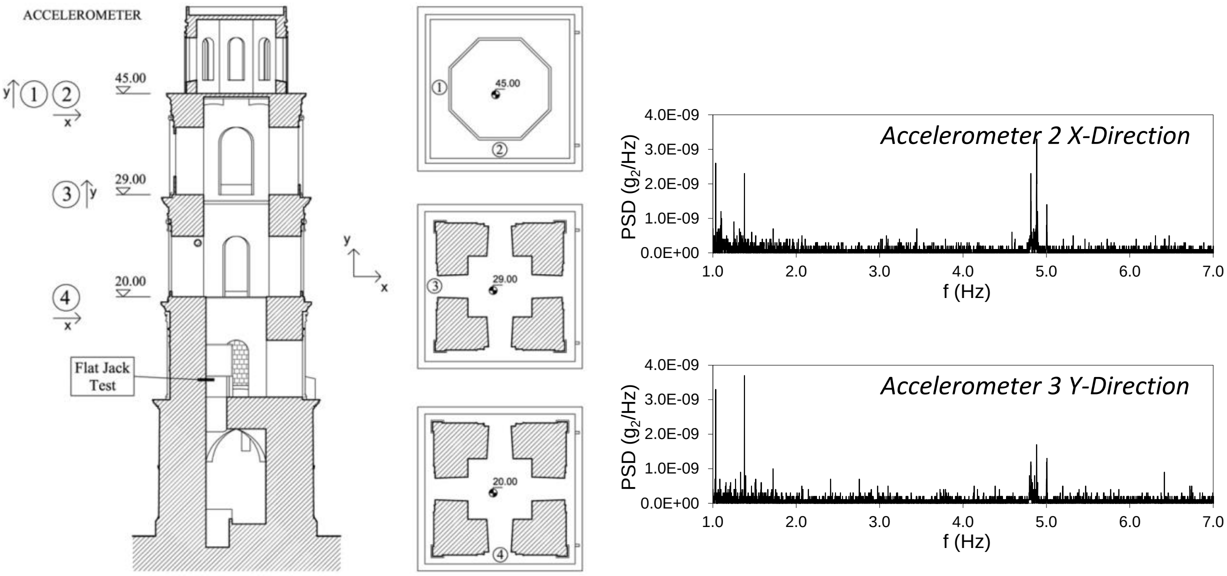



**Figure 4.** The bell tower of the Cathedral of Aversa: ambient vibration tests

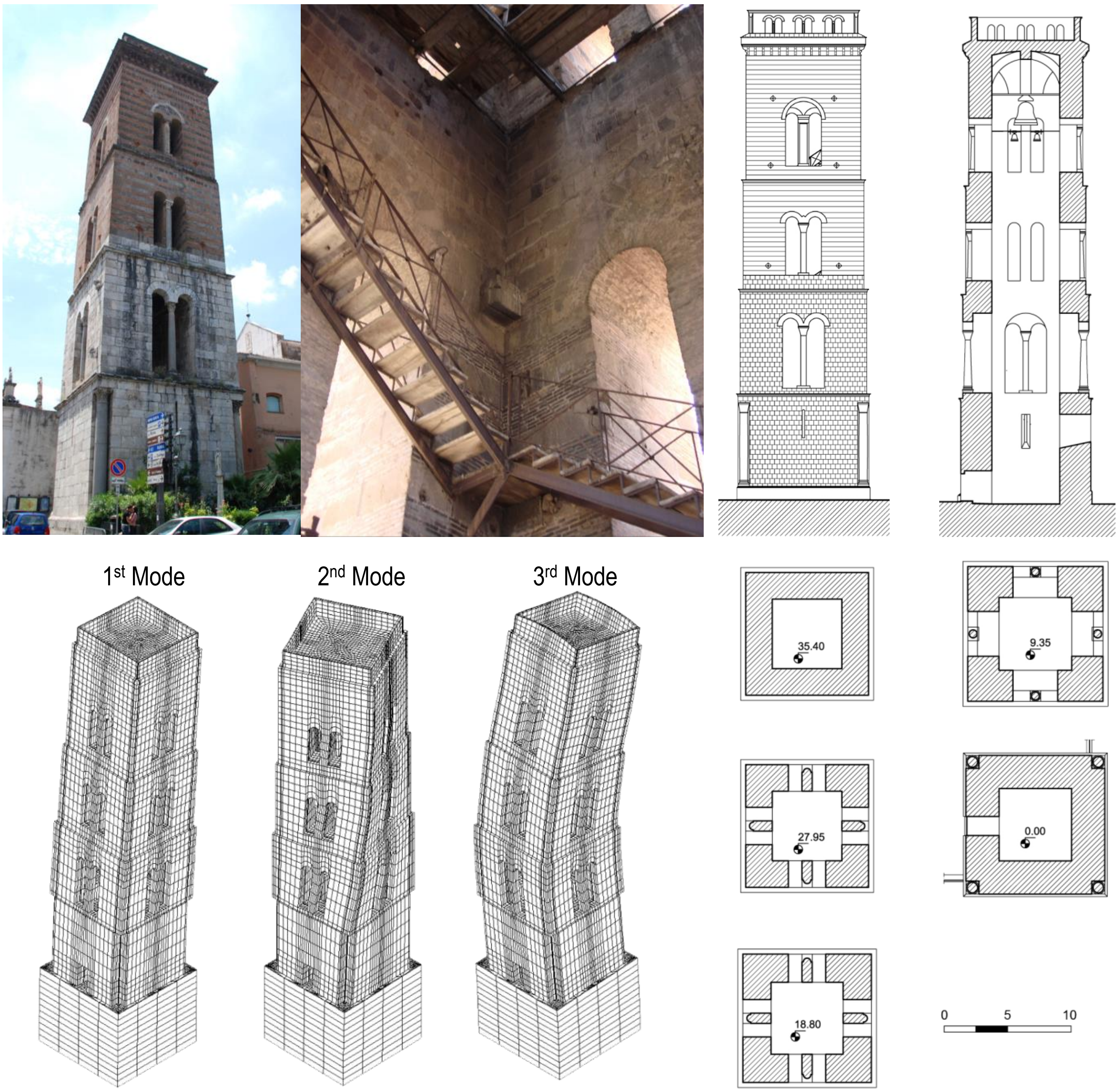



**Figure 5.** The bell tower of the Cathedral of Capua: general view, internal view, façade, sections, mode shapes

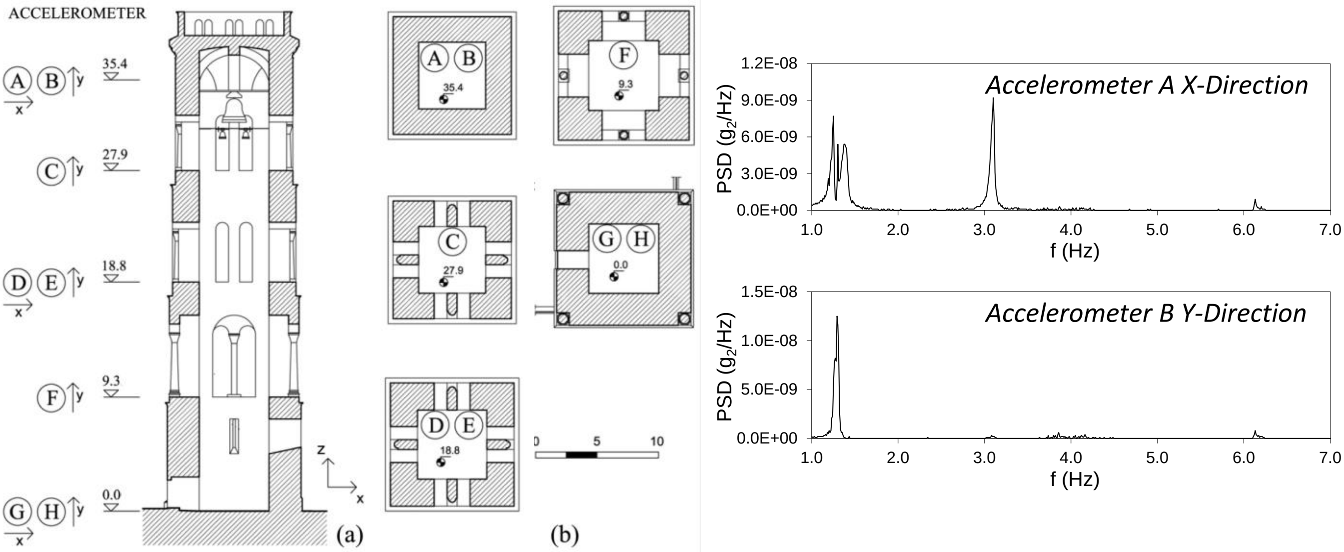



**Figure 6.** The bell tower of the Cathedral of Capua: ambient vibration tests

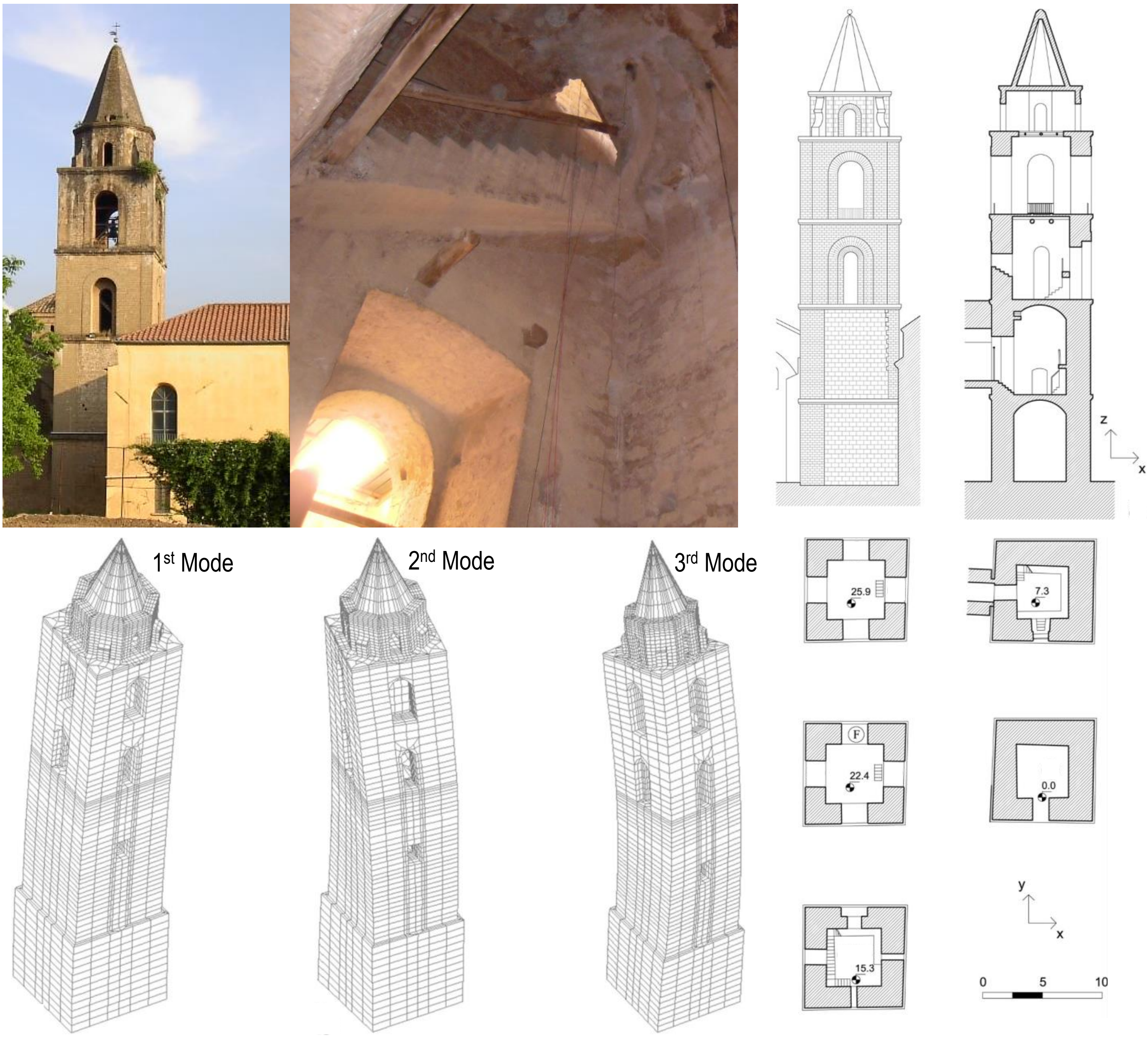



**Figure 7.** The bell tower of Santa Maria a Vico: general view, internal view, façade, sections, mode shapes

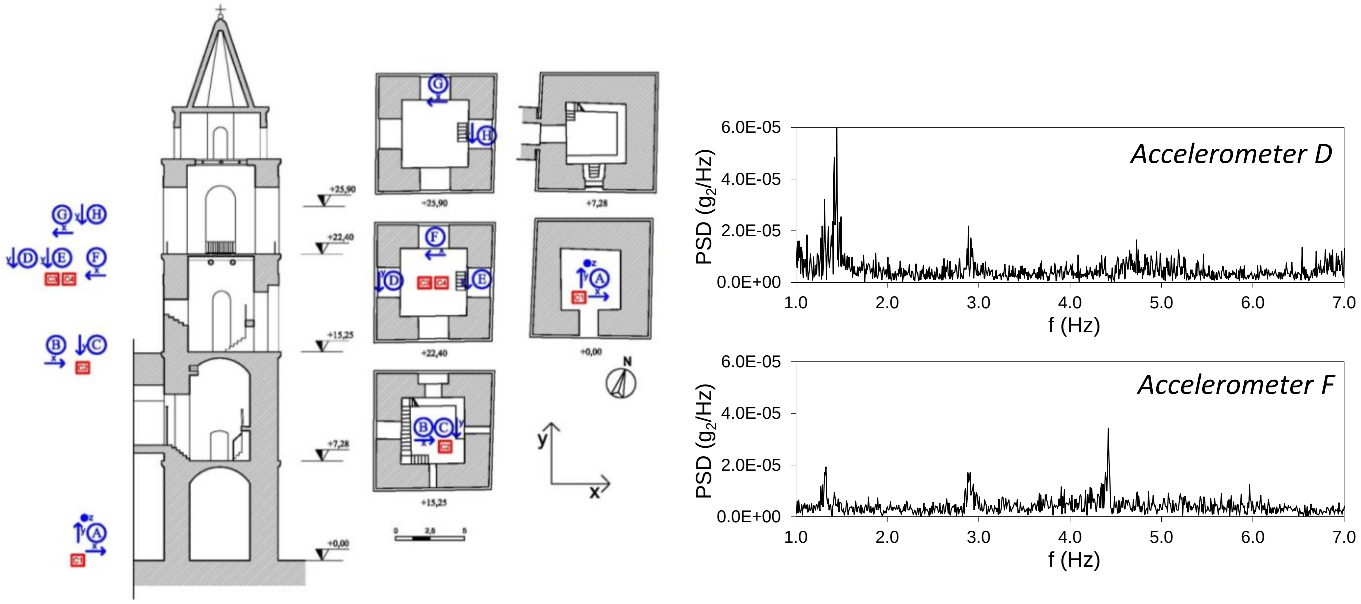



**Figure 8.** The bell tower of Santa Maria a Vico: ambient vibration tests

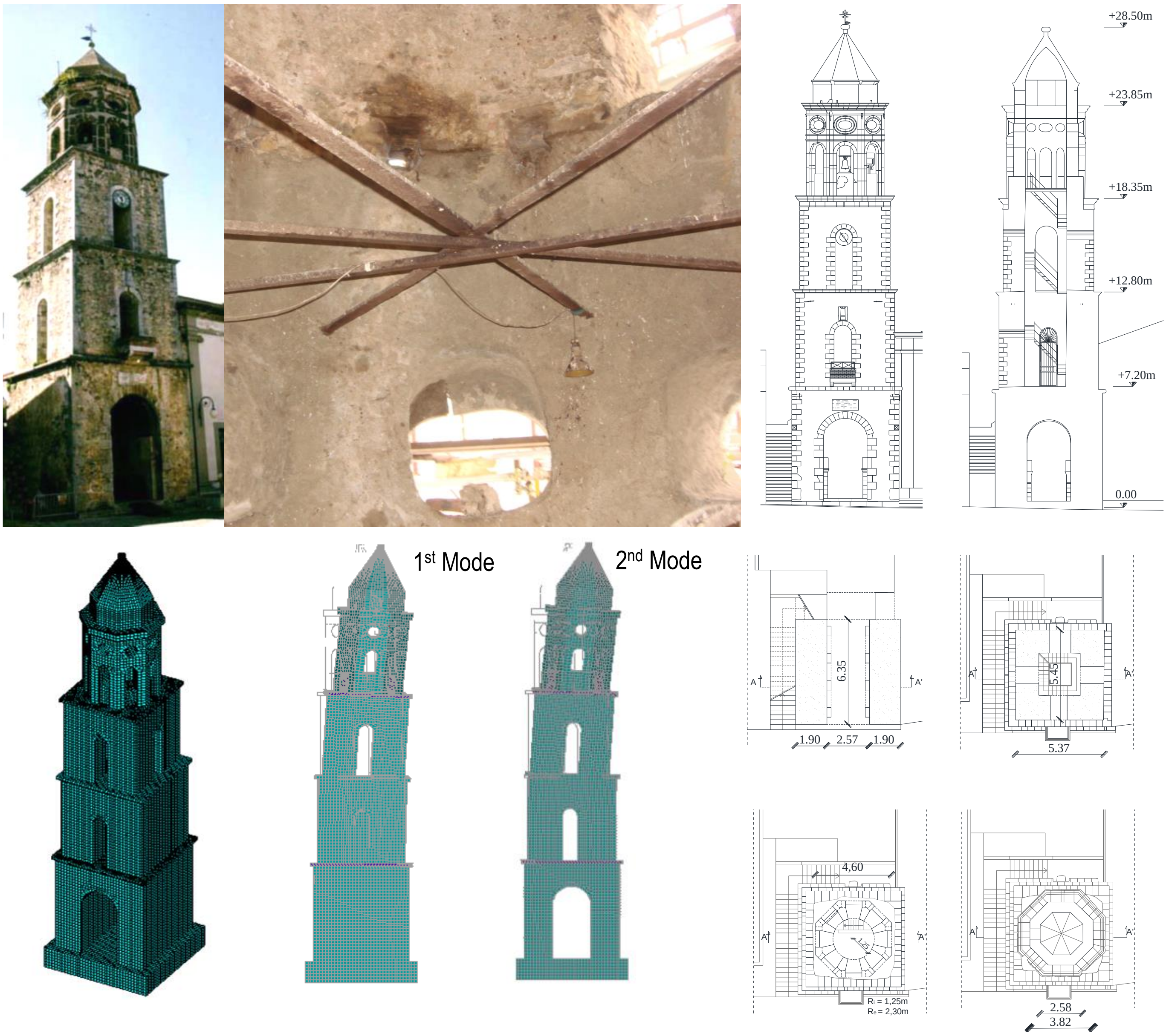



**Figure 9.** The bell tower of Torre Orsaia: general view, internal view, façade, sections, mode shapes

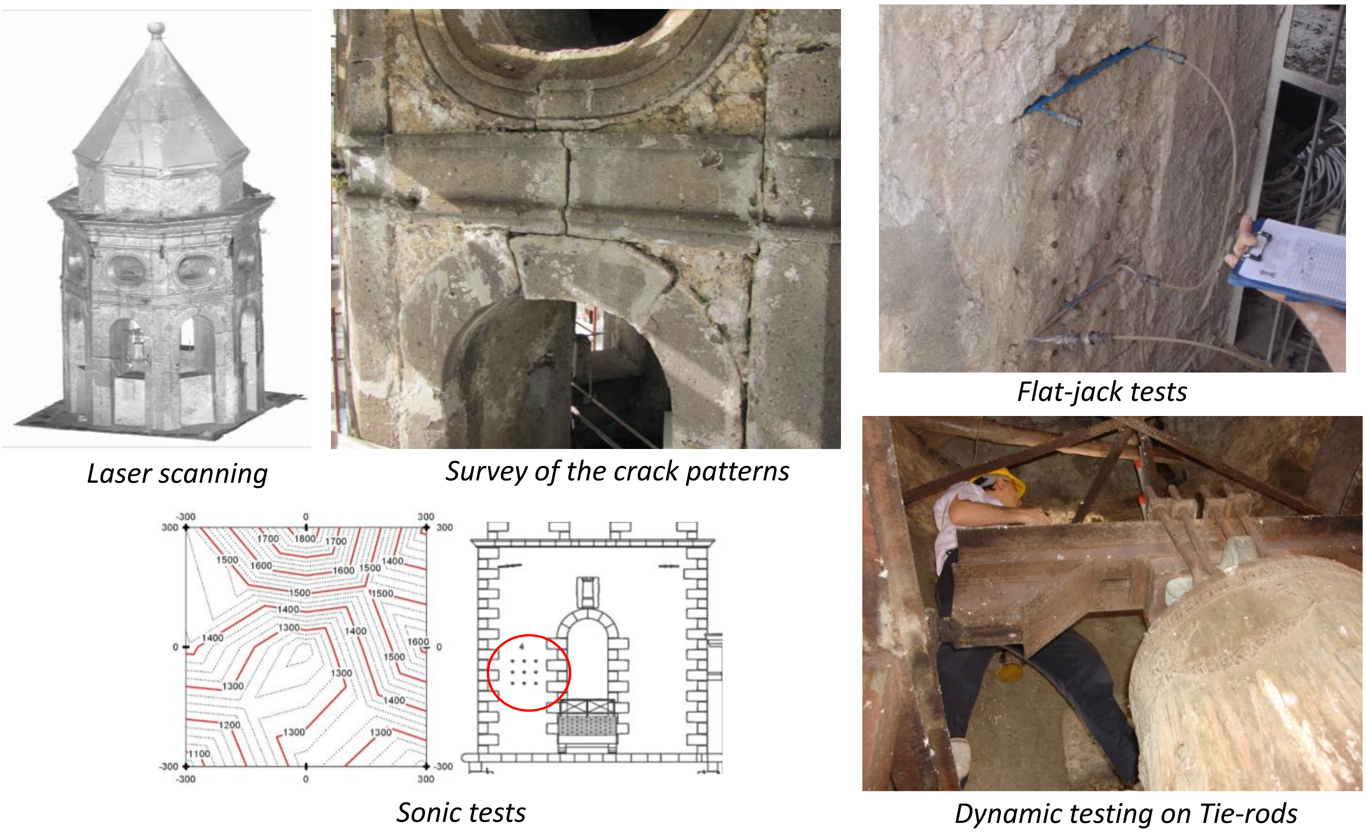



**Figure 10.** The bell tower of Torre Orsaia: laser scanning, crack patterns, flat-jack tests, sonic tests, dynamic testing on tie-rods

The windows, openings, and octagonal belfry are covered with squared grey tuff blocks. Horizontal iron tie-rods are placed to keep the walls from flexing outward. Investigations, on-site surveys, and tests were carried out on the tower including, geometrical survey, laser scanning survey, endoscopic tests, surveys of the crack patterns, sonic pulse velocity tests, flat jack tests, geognostic surveys, environmental vibration tests (Fig.10) (Ferraioli *et al.*, 2020)

## 3. MODELING and SEISMIC ASSESSMENT

### 3.1. FEM Tuning

The seismic behavior of masonry towers largely depends on their geometric properties and, particularly, their slenderness (i.e., height to base length ratio), the percentage of openings particularly in slender belfries, and the width of the perimeter walls. Moreover, it should be observed that the historical masonry towers are part of the urban area. Therefore, the constraints of the neighboring structures considerably affect the seismic behavior of the towers as well as their vulnerability. Another important aspect is the soil conditions that may play a key role, especially in the case of soil-structure interaction effects in soft soils. The modal parameters (i.e. mode shapes and natural frequencies) assessed via the environmental vibration tests were used to identify the unknown parameters of the numerical model. To update the initial model, the modal analysis was carried out and its results were matched to the experimental results to adjust the material properties, the geometry, and interaction with the adjacent buildings. The soil-structure interaction was modeled using linear elastic springs uniformly distributed in both vertical and horizontal directions. An iterative approach was followed and as a result, the values of all the unknown parameters (i.e., mechanical properties of materials, constants of the springs modeling the soil-structure interaction, constants of the springs modeling the constraints of the adjacent buildings). This iterative process was continued until a good match was observed between theoretical and experimental results.

### 3.2. Nonlinear Modeling

The seismic performance and collapse mechanism of the masonry towers when subjected to earthquake loading are considerably affected by the materials and construction techniques used. Moreover, the compressive stress of masonry due to the dead load, as well as the connection between the structural members may play an important role. Finally, the complex nonlinear behavior of masonry and its deterioration under cyclic loading should be accounted for. Often the compressive strength of masonry cannot be determined exhaustively through on-site destructive tests. Therefore, the data reported in the literature may be used along with the data that also complies with those available from other masonry towers similar for their main geometrical and structural characteristics. In this paper, the 3D nonlinear finite element analysis was carried out using the macro-modeling approach based on material homogenization. A model extensively applied in the literature was used, i.e., the perfectly elastic-plastic model based on the Drucker–Prager (DP) yield surface. The material properties of the model were determined by making sure that the circular cone yield surface of the Drucker–Prager model corresponds to the outer vertex of the hexagonal Mohr–Coulomb yield surface (Ferraioli *et al.*, 2017, 2018, 2020). The compressive strength of masonry was calculated as the mean value from the flat-jack tests if available. The tensile strength was considered as 1/15 of the compressive strength. The friction angle and cohesion were calculated based on the Italian Code (NTC-Guidelines, 2018; NTC-Instructions, 2019). The computer code LUSAS (2012) was used to perform the nonlinear analyses of the towers of Aversa, Capua, and Santa Maria a Vico, while ANSYS (2001) was applied for the tower of Torre Orsaia. The lateral restraint offered by the adjacent building was treated conservatively. Practically, it was considered in cases where the connection of the tower to the adjacent building is compressed, while it is neglected otherwise.

### 3.3. Seismic assessment

The seismic safety of the towers was evaluated based on the Italian Guidelines (2011) according to the level of analysis LV3 (global finite element analysis). According to the pushover approach provided by the Italian Code (NTC-Guidelines, 2018) two lateral load patterns (i.e., uniform distribution and first mode distribution) were considered in the nonlinear static analysis. This gives the capacity curve that plots the base shear as a function of top displacement. The last point of the curve corresponds to the structural collapse of the structure that is when occurring extensive cracking and crushing. In this case, a visible softening of the pushover curve should be observed. Even if this softening does not occur, the Italian Guidelines (2011) allow defining the ultimate limit state displacement in the range of 3–6 times the elastic displacement of the equivalent SDOF system. Moreover, the ductility ratio ($\mu$) equals the behavior factor ($q$) based on the hypothesis of equality of the maximum displacements. Thus, the conservative choice of $\mu$=$q$=2 was made centered in the range of 1.5–2.5 provided by Eurocode 8 (2004) for the behavior factor of unreinforced masonry buildings. The target displacement at the performance point was calculated using the capacity spectrum method (CSM) based on the inelastic demand response spectra originally proposed by Fajfar (1999) and then introduced in Eurocode 8 (2004) and Italian Code (NTC-Instructions, 2019). Fig. 11 compares capacity and demand in the ADRS (Acceleration-Displacement Response Spectrum) format. With increasing the peak ground acceleration also the target displacement increases. Thus, the peak ground acceleration may be amplified until the target displacement equals the life safety (LS) displacement. In this situation, the Inelastic Demand Response Spectrum (IDRS) intersects the Bilinear Capacity Spectrum (BCS) in the Life Safety performance point. The minimum value obtained from all the nonlinear static analyses is a measure of the actual capacity of the tower for the life safety (LS) limit state (i.e., capacity peak acceleration $a_{LS}$). The seismic demand is given by the reference peak ground acceleration ($a_g$) of the Italian Code (NTC-Guidelines, 2018) for the life safety (LS) limit state. Tables 1-4 show the seismic parameters of the elastic design response spectra of the site of each tower. The capacity to demand ratio gives the safety index ($\alpha_{LS}$) in terms of peak ground acceleration (Table 5). The results show that only for the bell tower of Torre Orsaia the safety factor is lower than 1 indicating that seismic retrofit is required in this case study.

**Table 1.** Parameters of elastic design response spectra Bell Tower of Aversa

| Parameter | S | DL | LS |
|---|---|---|---|
| $P_{VR}$ (-) | 0.81 | 0.63 | 0.10 |
| $T_R$ (yrs) | 30 | 50 | 475 |
| $a_g$ (g) | 0.042 | 0.055 | 0.116 |
| $F_o$ (-) | 2.379 | 2.353 | 2.455 |
| $T_C$ (sec) | 0.285 | 0.318 | 0.368 |

**Table 2.** Parameters of elastic design response spectra Bell Tower of Capua

| Parameter | S | DL | LS |
|---|---|---|---|
| $P_{VR}$ (-) | 0.81 | 0.63 | 0.10 |
| $T_R$ (yrs) | 30 | 50 | 475 |
| $a_g$ (g) | 0.042 | 0.052 | 0.113 |
| $F_o$ (-) | 2.418 | 2.405 | 2.579 |
| $T_C$ (sec) | 0.285 | 0.322 | 0.434 |

**Table 3.** Parameters of elastic design response spectra Bell Tower of Santa Maria a Vico

| Parameter | S | DL | LS |
|---|---|---|---|
| $P_{VR}$ (-) | 0.81 | 0.63 | 0.10 |
| $T_R$ (yrs) | 30 | 50 | 475 |
| $a_g$ (g) | 0.050 | 0.064 | 0.166 |
| $F_o$ (-) | 2.340 | 2.355 | 2.424 |
| $T_C$ (sec) | 0.286 | 0.313 | 0.372 |

**Table 4.** Parameters of elastic design response spectra Bell Tower of Torre Orsaia

| Parameter | S | DL | LS |
|---|---|---|---|
| $P_{VR}$ (-) | 0.81 | 0.63 | 0.10 |
| $T_R$ (yrs) | 30 | 50 | 475 |
| $a_g$ (g) | 0.037 | 0.047 | 0.116 |
| $F_o$ (-) | 2.446 | 2.443 | 2.521 |
| $T_C$ (sec) | 0.281 | 0.324 | 0.447 |

**Table 5.** Capacity peak ground acceleration - safety index

| Tower | X-Direction | | Y-Direction | |
|---|---|---|---|---|
| | $a_{LS}$(g) | $\alpha_{LS}$ | $a_{LS}$(g) | $\alpha_{LS}$ |
| Aversa | 0.192 | 1.41 | 0.189 | 1.39 |
| Capua | 0.165 | 1.46 | 0.167 | 1.48 |
| S. Maria a Vico | 0.187 | 1.13 | 0.187 | 1.13 |
| Torre Orsaia | 0.127 | 0.79 | 0.146 | 0.91 |

## 4. CONCLUSIONS

Many uncertainties lie ahead about the historic constructions including the mechanical properties of materials, foundation structures, soil-structure interaction, restraint of adjacent buildings, and so on.

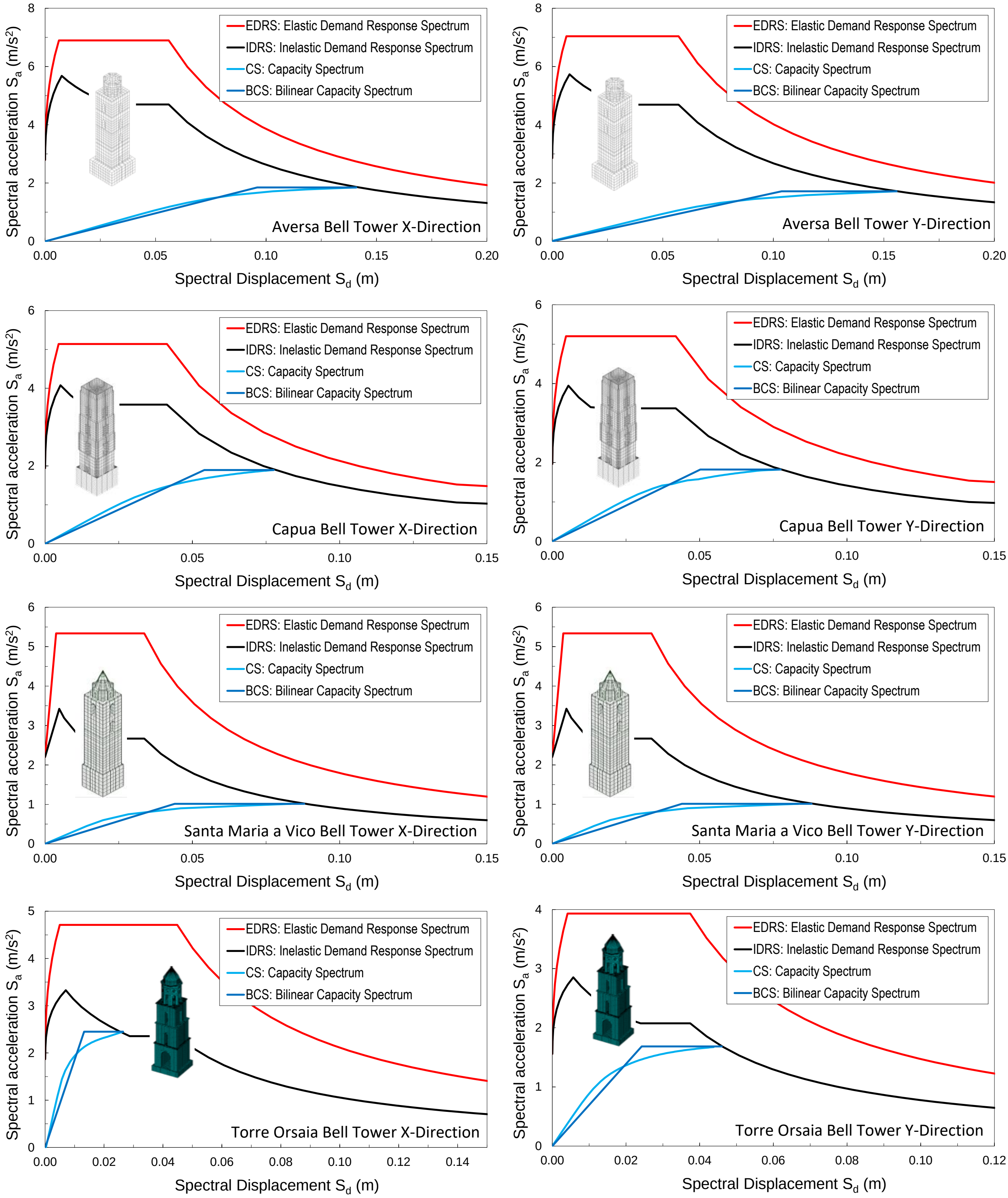



**Figure 11.** Comparison between capacity and demand in the acceleration–displacement response spectrum plane

This paper summarizes the experience gained in the development and implementation of investigations, on-site surveys, and tests on four historical masonry towers in Southern Italy. The experimental campaign included geometric surveys, laser scanning, endoscopic tests, sonic pulse velocity tests, geognostic surveys, flat-jack tests, ambient vibration tests, and dynamic tests on tie-rods. This experimental campaign made it possible to implement an accurate nonlinear model that was then used for global finite element analysis. The subsequent theoretical investigations were conducted to investigate the seismic vulnerability of the towers. The results showed that only the bell tower of Torre Orsaia exhibited a safety factor lower than one (i.e., structure to be considered unsafe). This is due to the choice of the most accurate level of analysis according to Italian Guidelines (2011) (i.e., level of analysis LV3) that allowed to obtain more effective and less conservative results.

**Author contributions**

Massimiliano Ferraioli: Study conception and design, Methodology, Analysis and interpretation of results, Writing-Reviewing and Editing.
Donato Abruzzese: Study conception and design, Methodology, Data curation, Investigation, Software, Validation

**Conflicts of interest**

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper

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**Cultural Heritage and Science**
https://dergipark.org.tr/en/pub/cuhes
*e-ISSN 2757-9050*

# Technologies for Architectural Restoration Works in Eighteenth Century Rome: The Birth of the Modern Scaffolding Practice

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**Keywords**
Scaffolding,
Building site,
Architectural Restoration,
St. Peter's Basilica,
Architectural treatises.

**ABSTRACT**

In 17th and 18th century Rome, building site technology depended on a consolidated empiricism rather than on formalized cognitive processes. Technologies inherited from the Imperial Rome continued to prevail, renewed by the experience of the greatest Renaissance and Baroque architects, and, above all, by of the colossal building site of the new St. Peter's. Starting from the second half of 17th century, the design of restoration scaffolding became indispensable for the conservation of the Vatican Basilica and its precious decorations, as well as for the conservation of the Pantheon, the Basilica of St. John in Lateran, and for other important monuments' maintenance, in Italy but also in some European nations. However, the development of scaffolding went almost unnoticed by the principal treatises on architecture, nor did it even find its way into technical handbooks. There are just a few exceptions in the contemporary literature on artistic techniques in 18th century. These works fit into the general context of the lavishly illustrated machine books, which had flourished during the 16th and 17th centuries, a genre, which gradually ended in the early 18th century, due to the changing attitude towards technology.

This paper clarifies the origins and the gradual improvement of scaffolding for restoration, indispensable for the maintenance of the architectural heritage. The scaffoldings designed in the early 18th century and their progressive improvement are analyzed. Among them, scaffolds used for the restoration of the domes of St. Peter's and the Pantheon assumed model authority in Italian and European restoration practice, and was celebrated in European carpentry manuals until the early twentieth century.

## 1. INTRODUCTION

In the mid-17th century Rome, building practice was finally perfected. This was due, among other things, to the development of an effective system for the economic and technical management of great Papal building sites and of a myriad of private new architectural constructions. This improvement took place at the same time as the slow progress of the new St Peter's Basilica in the Vatican's construction and thanks to the experimentation, it induced in the roman building "industry", both in the technical and administrative spheres. At the same time, building techniques and technologies expressed the unique specificity of the Roman context and its congenital relationship with antiquity, finding fruitful development in the many construction sites opened in the city. They became the major experimental laboratories for architects and workers of different origins. At the same time, the proven experience of the masters from Tuscany and Lombardy, and the fruitful cooperation between papal and private building sites were decisive in the early 17th century, and turned the Roman building practice into a recognized and appreciated model for construction techniques, material procurement, and organization of the workers and development of building technology. One of the fastest growing sectors is that of scaffolding for restoration, which is indispensable for the ordinary and extraordinary maintenance of Christianity's first Basilica.

Between 1700 and 1800, thanks to the inventions of a group of skillful carpenters from St. Peter's, scaffolding

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**Cite this article**

*Marconi M (2021). Technologies for Architectural Restoration Works in Eighteenth Century Rome: The Birth of the Modern Scaffolding Practice.* Cultural Heritage and Science, 2(2), 61-69

**Research Article**

**Received: 09/11/2021; Accepted: 22/12/2021**

for restoration was perfected and made effective for works also outside the Vatican. Their proven functionality extinguished in the first decades of the 20th century, when metal scaffolding came into use. Some of these, however, still carry the memory and characteristics of the wooden scaffolding used for the restoration of St. Peter's and the Pantheon, celebrated in European treatises and manuals until the 1920s.

From a methodological point of view, the information taken from the technical literature of the 18th and 19th centuries was compared with that found in the Vatican archives. These clearly illustrate the various types of scaffolding, their components, measurements, wood species used, as well as construction, assembly and dismantling techniques. Clear indications are also given on how scaffolding was used for restoration, both in large and small Roman buildings, from which it is now possible to trace the line of their gradual improvement and critical fortune in the European context.

## 2. The Restoration Scaffolding in the History of Construction and Building Technology

Construction machines, supporting beams and scaffolding in use in Italian Renaissance and Baroque construction sites have been widely investigated by scientific literature, as well as work organization, production and transport of materials and construction techniques. Although widely practiced in the modern age and documented in literature for only a few authoritative interventions in Roman buildings, scaffolding for ordinary and extraordinary maintenance remained undocumented for a long time, and they were only rarely the subject of architectural treatises.

This was because the knowledge of mathematics and the emergence of modern science have long been inevitable dividers between the intellectual and operational components of the construction profession. The only exception was the construction machinery, already included by Vitruvius in Book X of *De Architectura* and subsequently described by all the major Renaissance treatise writers. The correct execution of scaffolding is of fundamental importance for the good execution of the construction work, as well as restoration. Nevertheless, it was overlooked by Vitruvius and Leon Battista Alberti, but also by Andrea Palladio, Vincenzo Scamozzi and even by Philibert de l'Orme (De l'Orme 1570), which deals extensively with the subject of structural carpentry, as well as the characteristics and mechanical properties of the various wood species. There is a glaring gap in the illustration of the practice of building even in the famous *Encyclopédie* by Denis Diderot and Jean-Baptiste Le Rond d'Alembert (Diderot, D'Alembert 1751-1780). This may have been because in all building sites of the early modern age the construction of scaffolding was entrusted to the experience of artisans like bricklayers and carpenters. They were responsible for inventing new platforms, perfecting the ones already in use, and adapting each work of scaffolding to the specific needs of the construction, and later for restoration as well. Yet this important work did not deserve to be included in the intellectual treatment of architectural work, at least until the technical manuals became widespread. The importance of scaffolding for the success of the work was also well known to the supporters of the theoretical-intellectual component of the construction profession. In the mid-19th century, the French architect Emmanuel Eugéne Viollet-le-Duc (1814-1879) stated that «a well-designed scaffolding is one of the elements of the art of construction that best shows the intelligence and the good leadership of the project. One can judge the science of the builder by the manner in which he sets up the scaffolding. Well-built platforms save time and, by ensuring the workers' safety, permit them to work with greater regularity, method, and care». In 1871, the Milanese engineer Luigi Mazzocchi dedicated an entire treatise to wooden constructions, in which he included an extensive chapter on provisional works and an authentic tribute to the scaffolding designed by Nicola Zabaglia (1667–1750) (Mazzocchi 1871). He was a master carpenter of the of St. Peter's Fabric (the Papal institution in charge since 1506 for the construction and maintenance of the Vatican Basilica) and his astonishing inventions, as well as those of his successors, were considered virtuous examples of operational empiricism for a long time. Some of them were included in the main European treatises on construction up until the early 20th century. The long-lasting fortune of the Zabaglia's scaffolding shows that the scientific method did not make the empirical approach obsolete, but a gradual incorporation of theoretical precepts sanctioned the successful qualitative transition in the relationship between theory and practice even on the construction site. Zabaglia's activity includes a series of important interventions carried out among his forty-year service at the Vatican Institution, during the first half of the eighteenth century. At that time, the oral transmission of technical knowledge and operational empiricism progressively entered a crisis due to the rapid progress of scientific methods. The inventions of Zabaglia represented an authoritative model of cohesion between architectural theory, construction site experience, applied mechanics and restoration practices. Thanks to the Zabaglia's experience, scaffolding could be adapting to the specific requirements of restoration or maintenance works, also in the immense and variously articulated interior space of the St. Peter's. During the second half of 18th and all the 19th century, Zabaglia's partners and collaborators, still perfecting restoration scaffolding, ferried his inventions to the threshold of the 20th century, earning the celebration in the most important European carpentry treaties.

The numerous maintenance operations carried out in St. Peter's Basilica from the late 17th century until the early decades of the 20th century were formidable vehicles of development for the design of restoration scaffolding. They testify to the long-standing and never-discussed adherence to traditional operational practice. Reliable and safe, this practice made use - in construction as in restoration - of traditional materials, techniques and technologies tested by centuries of experimentation. The use of traditional building practices is documented in both routine maintenance and more demanding consolidation, rehabilitation and restoration works

The intuitive and perfect control of “contrast and balance of forces” allowed Nicola Zabaglia to test provisional apparatus and machines “well connected, strong and safe to work on any higher and more difficult site”. Therefore, his work represents a turning point for the development of construction technology: thanks to it, “there was no longer any need for Engineers and artists to direct and work on a scaffold, which cost the *Fabbrica* (St.Peter’s) a lot of money, and carpenters [...] became perfect in all works of mechanical engineering”. Moreover, Zabaglia's prodigious technical inventions have the potential for interesting adaptations to current construction and restoration practice.

The restoration work carried out in St Peter's between the eighteenth and early twentieth centuries is characterised by a substantial uniformity of procedures and practices. They derive from the widespread practice of “ripristino” (restore), not yet contaminated by the theoretical speculations of the mid-twentieth century, and entrusted to the operative wisdom of several generations of highly qualified building craftsmen, carpenters and masons, capable of assembling daring mobile scaffolding at the dizzying heights of the Basilica. The integration of fallen portions of stucco decorations, the replacement of stone elements, the reintroduction of some parts of cornices, the displacement of statues and furnishings, the replacement of columns, the restoration of the great Dome, including the consolidation with iron hoops (Marconi 2009; Dubourg Glatigny 2017), and the mounting of statues were possible thanks to the ingenious dexterity of skilled “sanpietrini” [of St. Peter’s] carpenters.

Traditional practices and techniques are therefore responsible not only for the material survival of St. Peter’s and for its impressive decorations, but also for the authoritative role played by the St. Peter’s Fabric in Roman construction field, at list until the middle 19th century (Marconi 2004). Its extraordinary efficiency in the organization of work force and its influence on the improvement of building techniques are strongly connected to the development of scaffolding, and related working spaces. The Vatican Basilica’s precious architectural space imposed severe restrictions to the design and functionality of restoration scaffoldings that had to be assembled and used without interfering with liturgical uses.

The temporary works employed by Nicola Zabaglia at St. Peter’s in Rome were published in the two editions of the *Castelli e Ponti* (literally “scaffolds and platforms”) (*Castelli e Ponti* 1743; 1824). The publications dates of the two editions neatly mark the end of the “old world” of show-off presentation books on technology, and the rise of the new scientific word of mechanical engineering. Zabaglia’s inventions have been described as the masterwork of a genius with innate and miraculous talents. However, a correct evaluation of his work is possible only through an in-depth analysis of his biography and the technical features of the scaffoldings conceived by him. Furthermore, the reasons that motivated the St. Peter’s Fabric to produce two editions of the book need to be analyzed. If in the 17th century, an “engineer” was defined as a person who could practice *ars* and *technè* (arts and sciences) with ability and keen intelligence, this title could have been assigned to the simple master carpenter Zabaglia, thanks to his uncommon professional ability, and to the importance of his inventions.

Zabaglia’s uncommon ability allowed him to achieve the exemption from the daily incumbencies on other workers and the privilege to use as own office. This was a private workspace, where he experimented mechanical prototypes from his inventive through models in scale, perfectly working, built with wooden sticks, production wastes of carpenters. The knowledge and the insightful control of the “contrast and balance of forces control” allowed Zabaglia to conceive and improve standing and flying scaffoldings, permanent or mobile, cheap, strong, safe and suitable for both routine maintenance, and extraordinary restoration procedures. During his long career, Zabaglia successfully completed several technical undertakings, characterized by the use of scaffolds composed of simple wooden elements that could be controlled by means of a network of ropes, driven by hoists. Thanks to his incredible intuition and with apparent ease, Zabaglia brought to perfection these devices, improving their components, simplifying their warp and turning them into movable devices that could be quickly placed where they were needed. Consequently, the St. Peter’s Fabric was able to reduce execution cost and time; moreover, downtime was also shortened, as it was no longer necessary to dismantle and reassemble all the components. Thus, shoring, scaffolds and machines became more effective, and even complex interventions could be carried out in a short time, saving many materials, as well as ensuring the successful completion of the work.

Among the most relevant works done by Zabaglia in the Vatican Basilica since 1686, and celebrated in the *Castelli e Ponti* book, are the scaffolding used for the restoration of the porch vault’s stuccoes (pl. XXI_fig.1), those for maintenance of the minor domes and for the main dome's pendentives, and the one settled on the molding of the main dome for the mosaic restoration (*Castelli e Ponti* 1743, pl. XXVI). From the constructional and functional point of view, the devices built for the restoration of the main nave's vault (pl. XXV_fig.3), and the multi-levels platforms so-called "Ponti Reali" (*Castelli e Ponti* 1743, pl. XXVIII) are not less interesting.

Comparable attention characterizes the project for scaffolding to clean the 30 meters high Berninian baldachin, and the installation of the fifty statues on the straight wings of Saint Peter’s Square. For this work, in 1703, Zabaglia experimented a perfected version of the “antenna”, a big wooden tower crane equipped with a mobile platform, which allowed him to reduce costs and times of work, closed in just three months. Zabaglia was also a protagonist of the well-known consolidation of the main Dome under the supervision of Giovanni Poleni and Luigi Vanvitelli since 1743, the same year of the *Castelli e Ponti* first edition.

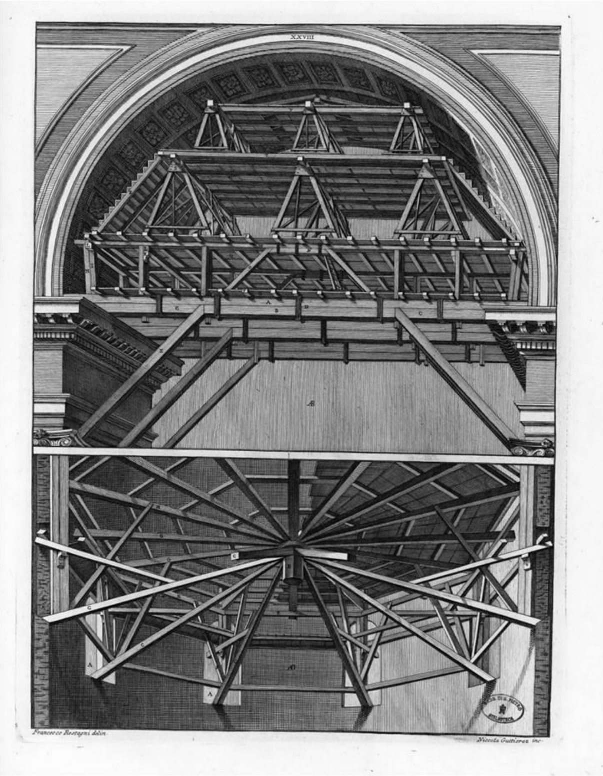

**Figure 1.** Scaffolding (so called "Ponti Reali") by Nicola Zabaglia for restoration works at the domes of St. Sebastiano and St. Michele in St. Peter's. 1721(*Castelli e Ponti* 1743, pl. XXVIII).

Zabaglia's talent, which revived the ancient operative pragmatism in face of the young and ongoing scientific advancement, legitimated by the celebrate volume dedicated to him, and his achievement was turned into a myth. His scaffolding share a constant care for the conservation of wall surfaces, the safety of workers and the reuse of materials. They represented such a remarkable progress in comparison with the usual building mechanical devices that they would be used without interruption up to the first half of the 20th century and abandoned only following the introduction of modern metal scaffolding. This celebratory publication and user manual at the same time, ordered by St. Peter's Fabric was inspired by the contemporary French and German manuals, in order not to lose its memory and to keep it for the next artisans generations. However, Renaissance treatises' influences still affected this volume, as the dual Italian and Latin text proves. *Castelli e Ponti* was responsible for transmitting ancient technical knowledge and for renovating the centuries-old supremacy of the Church, made weak by radical political and social changes. Thanks to this well-known compendium, Zabaglia's fame crossed the Italian borders and his inventions were recognized in carpentry treatises all over Europe. As far as the representations of the scaffoldings are concerned, the scenographic perspective tools were used to increase the magnificence of the machines and to impress the reader, as in the classical presentation books of the Renaissance age. Nevertheless, the plates excel by minute detail, enabling the reconstruction of the devices from their perspective depictions. In order to put Zabaglia's inventions in a dignified and worthy context, the editors of *Castelli e Ponti* enriched the publication by a selection of plates representing the famous translation of the Vatican obelisk, carried out in 1586 by Domenico Fontana (Fontana 1590).

*Castelli e Ponti* was a concrete proof of the Vatican building authority and of the great intellectual, technical and economical sources exerted in the construction and conservation of the St. Peter's, but also in defense of the Christian doctrine. Thus, technics spread, and political propaganda subtended to the promotion of Zabaglia's masterpiece, as an emblem of traditional but unfading know-how, comparable with the most advanced science's developments. The volume is divided into three parts. In the first one, tools, machines and work utensils ordinarily used by masonry workers and carpenters are illustrated (*Castelli e ponti* 1743. pls. I-XVII). The second section explains the scaffoldings designed by Zabaglia for maintenance and restoration practices (*Castelli e ponti* 1743. pls. XVIII-XXXVI). In its turn, this part is arranged in two clearly distinct partitions: the first (*Castelli e Ponti* 1743. pls. XVIII-XXX) includes scaffoldings for painters, plasterers and mosaicists, then also the ones necessary to bricklayers, carpenters and stonecutters. The second partition (*Castelli e Ponti* 1743. pls. XXXI-XXXVI) embraces scaffolds and platforms for diverse interventions, outside the Vatican also. The third and last section re-proposes the correct sequence of the plates illustrating the Vatican obelisk's transportation (*Castelli e Ponti* 1743. pls. XXXVII-LIV). The picture of the scaffolding built by Tommaso Albertini (*Castelli e Ponti* 1743. pl. LV), a skillful Zabaglia's assistant, to replace a similar structure previously made by Zabaglia for the restoration of Sts. Simeone and Giuda's tribune, concludes the volume. The sequence of the plates supports both the understanding of the individual device, and the promotion of the publication's general aims. The first plates are devoted to the tools for wooden works, connection devices and ropes knotting. These plates, through the graphic composition and the frameworks depicted, evoke some French compendia, edited in the previous century, as those by Mathurin Jousse (Jousse 1627), republished in 1702 in Paris. The accuracy of work tools, ropes, wooden grafts, and lifting machines illustrations (*Castelli e Ponti* 1743. pls I-VI) shows their consistency and typology, underlining their origins in the traditional equipment of masonry workers, stonecutters and carpenters. The detailed description of the specific techniques is also extended to materials, measures and weight, with the evident aim at demonstrating the practical aspects. The order of devices and tools, remarked in the illustrating captions sequences, follows a criterion based on the employment context.

*Castelli e Ponti* was conceived as a celebrative treatise as well as an operative handbook, destined to perpetuate the precious work of the carpenter masters and restoration techniques carried out by the St. Peter's Fabric and by its highly qualified technicians. Indeed, Zabaglia became the progenitor of a valuable group of St Peter's craftsmen. Due to their indubitable technical supremacy, they were even committed to solve difficult technical issues in worksites outside the Vatican, during the whole next century.

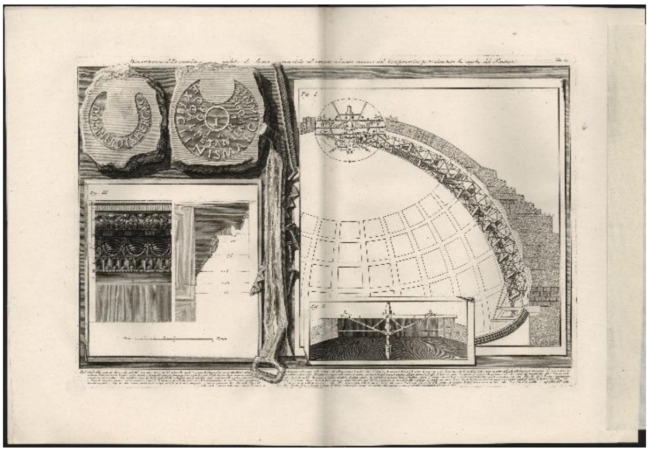

**Figure 2.** Scaffold designed by Giovanni Corsini and Tommaso Albertini for the Pantheon's dome restoration. 1756 (Piranesi 1790, pl. 29).

Among them, the rotating scaffolding design in 1756 by Giovanni Corsini and Tommaso Albertini (both contributors to Zabaglia) for the Pantheon's dome restoration is undoubtedly the most relevant. It is a mobile scaffold, operated by winches, and sliding on the Dome's frame. The scaffolding's structure was in the form of a spherical segment, anchored to a mechanical joint mounted in the free space of the dome's "oculus".

The followers of Zabaglia never formed a real school, but his teaching to some of the most brilliant laborers of the *Fabbrica* effectively guaranteed the transmission of his valuable technical knowledge. Notably, Giovanni Corsini, Angelo Paraccini and Tommaso Albertini contributed to the enhancement of the building practice and technology, so that some of their scaffoldings were included in the second edition of *Castelli e Ponti*, published in 1824. Tommaso Albertini, supervisor of the St. Peter's Fabric from 1773 until 1787, built the scaffolding for the restoration of the secondary dome of St. Gregory's (*Castelli e Ponti* 1824. pl. LVI), as well as the scaffold built for the restoration of the barrel vaults of the nave, replacing Zabaglia's solution (*Castelli e Ponti* 1824. pl. LV) Fig.3.

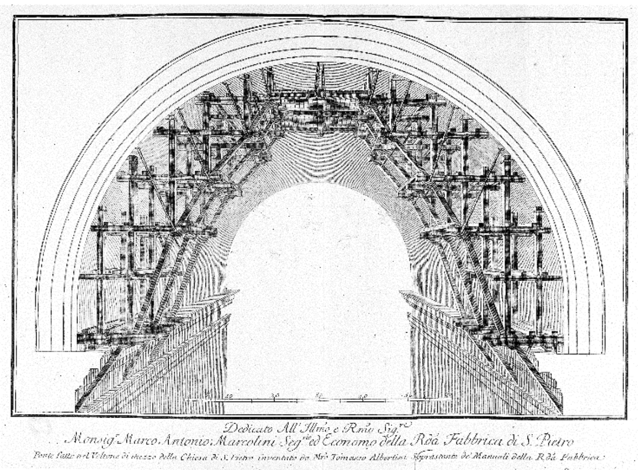

**Figure 3.** *Castelli e Ponti* 1743, pl. LV. Scaffolding by Tommaso Albertini for the restoration of the nave of St. Peter's, Rome.

The flying scaffolding for the restoration of the inner shell of the main dome of St Peter's, built by Tommaso Albertini, is equally interesting. Its profile designed to match the shape of the dome by the overlapping of different working platform; at its upper end it is anchored to three long poles fixed on the lantern's basis, which is connected with long inclined scissor braces (*Castelli e Ponti* 1824. pl. LVII) Fig.4.

During the French invasion of Rome (1798-1799) and the following occupation of the Church State in 1808-1812, papal authority and Vatican secular tradition entered a deep crisis. The challenge to establish a modern technical education, founded on scientific principles, endangered the technical primacy of the of St Peter's Fabric. New teaching criteria were imposed on all training sectors, including the professional and the craft ones. It was demanded to reorganize these according to the exemplar French model of the Schools of Arts and Crafts, enforcing openness towards the scientific progress that the Church could no longer postpone.

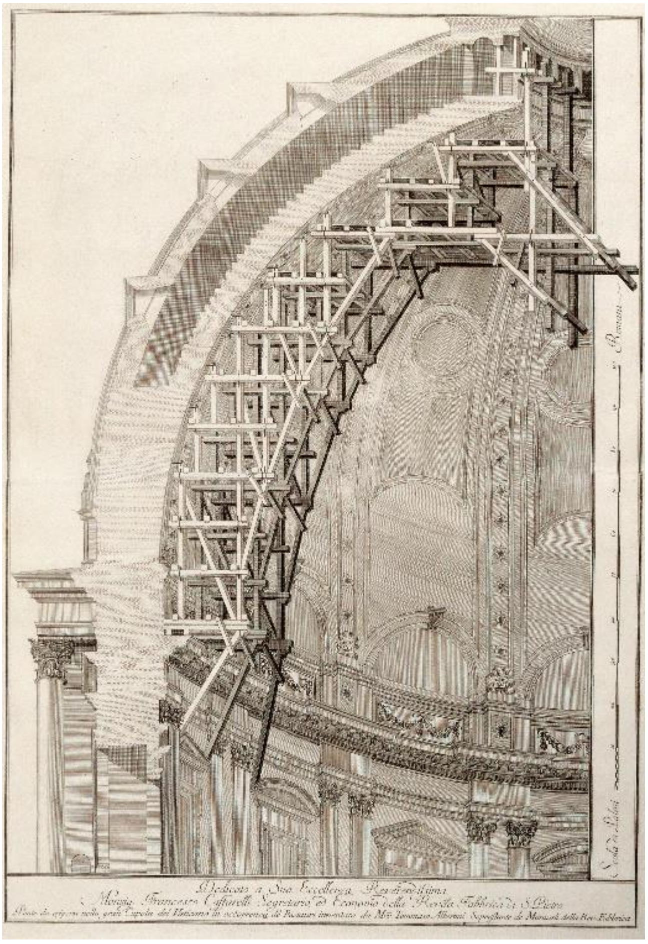

**Figure 4**. *Castelli e Ponti* 1824, pl. LVII. Scaffolding for restoration work of interior decoration of St. Peter's dome by Tommaso Albertini.

The second edition of the oeuvre concerning the work of the master carpenter Nicola Zabaglia saw the light at the end of this dark period. It was published in 1824 and includes 62 engraved copperplates and 48 pages with explanatory texts. A dedication to Pope Leo XII, an introduction by the treasurer of the St Peter's Fabric, bishop Castruccio Castracane, and a biography of Nicola

Zabaglia by the lawyer Filippo Maria Renazzi (1745-1808) accompany the work.

Among the new plates, some interesting technical inventions appear. Pietro Albertini, Tommaso’s son, designed the great scaffolding used to restore the stuccos in the main nave of St Peter’s Basilica (*Castelli e Ponti* 1824. pls. LVIII-LX). The impressive scaffold lifted to the nave’s cornice with six winches on 26 November 1773. This framework, acknowledged for its reliability, safety and respect for masonries, floors and ornamentations, stands out among the most brilliant mobile scaffoldings. The plates show not only the massive and safe structure, but also the method and the equipment for its installation, like the iron crowbars used to assure the anchoring. This framework was composed of eleven shelves, covering a whole sector of the vault.

**Figure 5.** *Castelli e Ponti* 1824, pl. LX. Installation of a moving restoration scaffolding designed by Pietro Albertini in the nave of St. Peter’s in 1773.

The flying scaffolding for the restoration of the inner shell of the main dome of St Peter’s, built by Tommaso Albertini, is equally interesting (*Castelli e Ponti* 1824. pl. LVII) Fig. 4. Its profile is designed to match the shape of the dome by the overlapping of different working platform; at its upper end it is anchored to three long poles fixed on the lantern’s basis, which is connected with long inclined scissor braces.

During the 19th century, other experts from the St Peter’s Fabric and Roman technicians continued to perfect the design and structure of the scaffolding for the restoration. The greater attention paid to the safety of the workers, the adaptability to the complex spatial geometry of Roman architecture of different periods and types, the speed of assembly and disassembly, the economy and reuse of the various components, whether wooden or metal, were the main lines of improvement.

The St Peter’s Fabric continued to be a fertile laboratory for experimentation, given the Basilica's pressing maintenance needs and its extraordinary uniqueness in terms of size, function and space.

The scaffolding designed at the end of the 19th century by St. Peter's superintendents, Enrico Donnini and Ercole Scarpellini, for the restoration of the main nave and the great dome, attest a substantial continuity with 18th-century practice (Fig. 6). At the same time, they introduced a design methodology that would lead to the invention of joint tube scaffolding. The scaffolding built in 1863 by Enrico Celso Donnini for the restoration of the gilded decorations in the Vatican dome perfected the one designed between 1816 and 1818 by Angelo Paraccini, later also used for the replacement of the lead sheet covering.

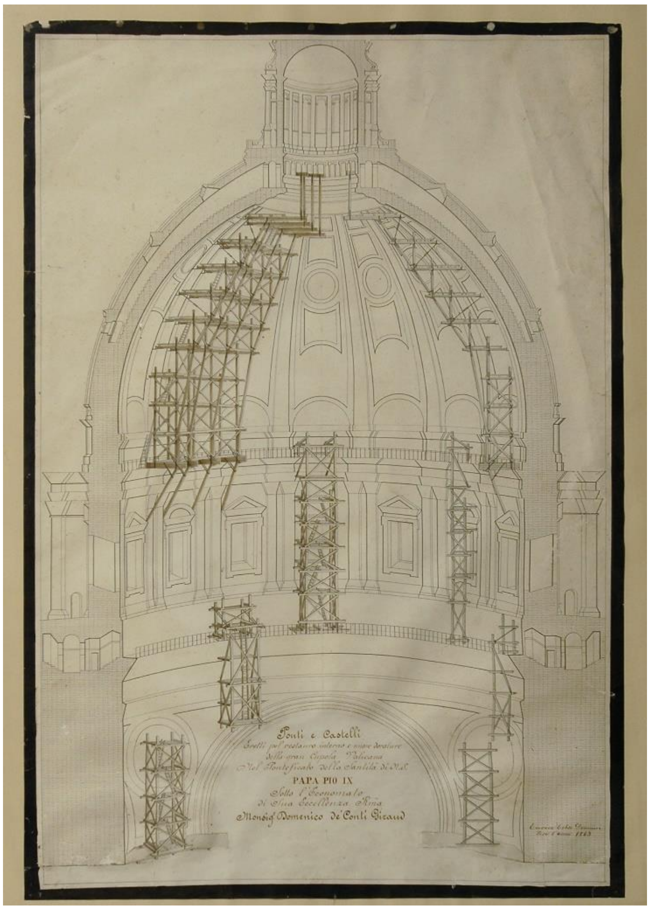


**Figure 6.** Enrico Celso Donnini, Scaffolding for restoration and new gilding work of St. Peter’s Dome. 1863. (Vatican City. @St. Peter’s Archives).

At the end of the 19th century, Ercole Scarpellini designed a colossal, multi-shelf, light and strong aerial scaffold with a span of 26 meters and a weight of approximately 9 tons (Fig. 7). This scaffolding was used to restore the stucco and gilding on the vault of the nave, damaged by a powder keg explosion in 1891 and the earthquake of 1895. On 16 November 1897, in just one-hour’s work and with the help of three winches, ropes and hand-pulls, the colossal scaffolding was erected on the big cornice of the main nave of St Peter's, some 30 metres above the ground. From the vault's setting two diagonal components supported the upper deck of the scaffolding, to which, in turn, the floorboard was fixed. Four levels of scaffolding allowed workers and artists to work comfortably and safely. The scaffold was stabilized by strong iron brackets and stiffening wooden elements placed on the diagonals. No invasive anchoring to the wall structure was required. As the previous prototype designed by Zabaglia, and later perfected by his successors, Scarpellini's "arched castle" could slide along the cornice thanks to soap-smeared slides, pulls and pulleys. It was described by contemporaries as a "bold work of art so well-conceived, worthy indeed of the artistic traditions of the first temple of Christianity" (Marconi 2015. 49-65).

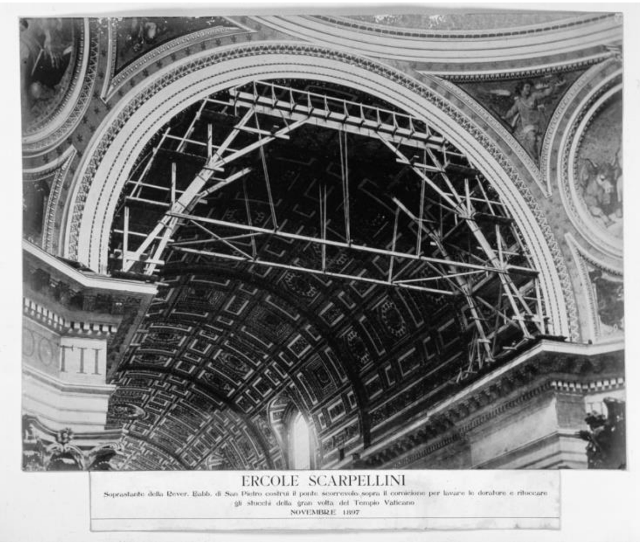


**Figure 7.** Scaffold design by Ercole Scalpellini in 1897 for the restoration work in the vault of the main nave of St. Peter's (@St. Peter's Archives)

From the point of view of critical acclaim, Zabaglia's teaching, developed and perfected by his collaborators and epigones, reached European treatises and manuals during the 19th century. In particular, technical literature and northern European carpentry manuals celebrated, the provisional apparatuses designed for the restoration and maintenance of the great Roman edifices, primarily the Pantheon and St. Peter's, re-proposing their shapes and constructional characteristics also in the explanatory graphic tables.

The first echoes of Zabaglia's work in later publications appears in the sumptuous treatise by Giuseppe Antonio Borgnis (Borgnis 1818-1820). References to Zabaglia's work and to the volume Castelli e Ponti appear in the two most influential Italian textbooks on construction and technology from the first half of the nineteenth century by Nicola Cavalieri San-Bertolo (Cavalieri San-Bertolo 1826-27) and Giuseppe Valadier (Valadier 1828-39).

Stefan Holzer affirms that Cavalieri San-Bertolo refers passingly to Zabaglia in his chapter on construction machinery and architectural procedures, and Valadier, in his turn, relied on the layout of some Zabaglia's plates as a model for his plates. Beyond these early Italian references to the first edition of Zabaglia, "the work received virtually no acclaim". References to scaffolding designed in the St Peter's Fabric do not appear in the works of Jean-Baptiste Rondelet (Rondelet 1802-1810) and Jean-Charles Krafft (Krafft 1805). According to Holzer, this is probably due not to his ignorance of Zabaglia, but rather might be attributed to the fact that all material referring to temporary works such as scaffolding, centers and lifting gear was excluded from his specialized *Traité des échafaudages*, which was printed only posthumously (Krafft 1856).

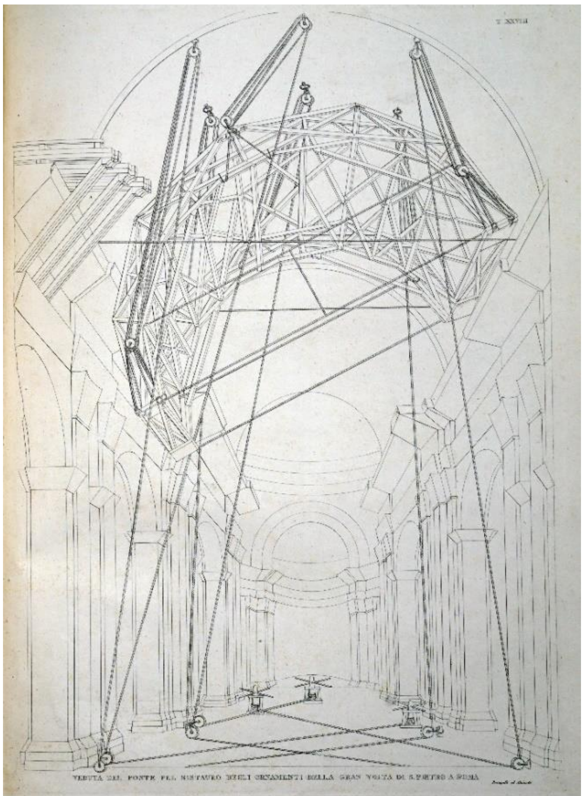

**Figure 8.** Pizzagalli & Aluisetti 1827, pl. XXVIII, showing the installation of the 18th century scaffolding by Pietro Albertini in the nave of St. Peter's, from Zabaglia 1824, pl. LX (here in Fig. 4).

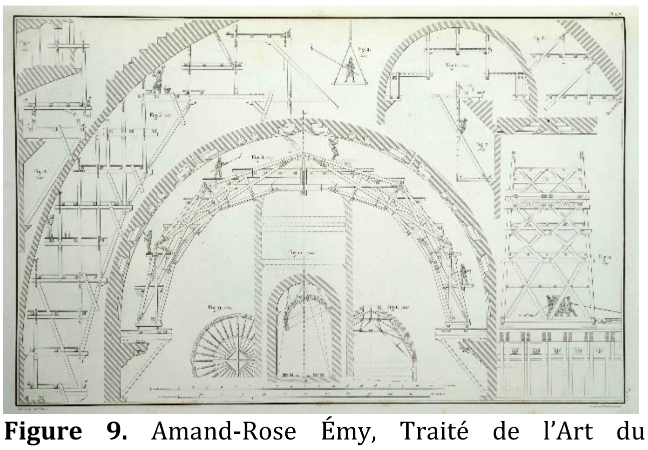

**Figure 9.** Amand-Rose Émy, Traité de l'Art du Charpentier (1837–41). Plate 127, showing scaffoldings for the restoration of the dome and the nave of St. Peter's, Rome, after Zabaglia 1824

However, it should not be ignored one fact decisive in understanding the reasons for the lack of success of the first edition of *Castelli e Ponti* in the treatise, whereas the operational success was immediate and prolonged: the first edition of the volume was published in 1743 in a small number of copies and they soon sold out. For a long time no copies were available for sale. The second edition, published in 1824, was reprinted from a rare 1743 copy that the *Fabbrica* itself was forced to buy at auction (Marconi 2015. 67-92). The circulation of the first edition in Europe was therefore very limited and could not influence the drafting of the carpentry and construction manuals of the time. However, there are some exceptions. The scaffolding for the restoration of the great dome of St Peter's, for example, had been included into the series of engravings by Gabriel-Martin Dumont (Dumont 1763).

## 3. CRITICAL FORTUNE AND RELEVANCE OF 18TH-19TH CENTURY WOODEN SCAFFOLDING

The second edition of *Castelli e Ponti*, by contrast, garnered much greater attention and was rapidly exploited in other publications; its critical fortune was conveyed by affirmation of technical schools and by the spread of scientific research achievements and its applications to the building site. Outside Italy, Amand-Rose Émy (Émy 1837-1841) introduced Zabaglia's work into the scope of the principal textbooks on carpentry. This publication was soon to become the reference text for the French-speaking world. Émy was most impressed by the Albertini scaffoldings for the St. Peter's nave and dome. He calls the scaffolding for the dome of St. Peter's "the most remarkable flying scaffolding", and depicting it, together with the scaffolding for the nave. The Albertini scaffoldings reappear in the carpentry treatise by Jean-Charles Krafft (Krafft 1856), but without any reference to the source. Later, some Zabaglia scaffolds were taken up again in another English carpentry manual, published by Edward Lance Tarbuck (Tarbuck 1857-1859). Here, with explicit reference to Zabaglia, Albertini's scaffolding for the nave of St. Peter's and Campanarino's rotating scaffolding for the Pantheon are again united in a single plate, both in orthonormal projection.

The last trace of Zabaglia within later technical literature is included a comprehensive collection of loose plates showing the Albertini's scaffolds, published by the Viennese contractor and carpenter Andreas Baudouin in 1908. When Baudouin's collection was reprinted in an abridged version in 1926, this plate was omitted, and the reception of Zabaglia came to a close (Baudouin 1908).

In the first half of the 20th century, the history of large wooden scaffolding gradually came to end. After the Second World War, the first metal scaffolding appeared in St Peter's too, which was lighter, reusable, flexible, cheap, safe and easier to maintain.

## 4. CONCLUSION

The inventions of Italian and European carpenters, which have made the history of modern technology, finally gave way to tube-joint scaffolding with prefabricated frames, and multidirectional or multi-storey scaffolding with prefabricated uprights and transoms. They were also adapted to restoration work. These require technical, formal and functional specifications for the protection of historical masonry and decorations. They must also be able to be suspended so as not to burden the structure and at the same time guarantee the usability of the building, specifications already known to the St.Peter's carpenters. The validity of their ideas has filtered into the current version of the devices they invented. A mention may be made of the special scaffolding built by EuroEdile (scaffold builders company) in 2005 for the conservation restoration of the Pantheon's dome. The scaffold adopts the spherical wedge shape already used in the 18th century and in the restorations of the 1920s, but updated in terms of components and technical specifications: 53,000 kg in weight for 43 m in height, 35 quintals of girders for the base, 500 m2 of worktops, 20 m3 of wooden boards, 48 dampened reinforced wheels, 6 of which are motorised and controlled by software. With this scaffolding, it was possible to leave about 70 % of the dome's surface free to allow visitors to enjoy the monument. These characteristics are similar to those of the scaffolding designed by Corsini and Albertini in the mid-eighteenth century. It would be interesting to try to adapt them to contemporary regulations and technology in order to verify their relevance and the possibility of new, certainly effective, and use potentialities.

## ACKNOWLEDGEMENT

The author would like to thank for the fruitful discussions on the research topic presented here Prof. Stefan M. Holzer, ETH Zürich and Prof. Donato Abruzzese, University of Rome Tor Vergata.

### Author contributions

All contributions belong to the author in this paper.

### Conflicts of interest

There is no conflict of interest between the authors.

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**Cultural Heritage and Science**
https://dergipark.org.tr/en/pub/cuhes
***e-ISSN 2757-9050***

# The Transmission of the Technology for Gothic Masonry Vaulting to Greece, in the Case of Saint Sophia in Andravida, Elis

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**Keywords**
Technology transmission, Gothic, vaults, Andravida, Morea.

**ABSTRACT**
The church of Saint Sophia in Andravida, built around the mid-13th century in Elis, Western Greece has its still-remaining apse roofed in ribbed cross vaults. Built by the Frankish Princes of Achaia who occupied in the 13th and 14th century an area dominated by the native Byzantine architecture, Saint Sophia shows the great effort and attention paid in transferring new architectural forms and technology into a politically and culturally alien environment. This paper discusses the vaults' construction and structural behaviour and explores questions around the technology transfer mechanisms from Western workshops. Although efficient, the vaults appear rather basic, and conservative compared to the contemporary endeavour to gradual disintegrate the envelope in Gothic architecture in Western Europe. On the whole, the analysis of the geometry, construction and structural performance showed a well-executed design with direct local input only at the construction stage.

## 1. INTRODUCTION

Following the fall of Constantinople in 1204 the lands of the Byzantine Empire were divided between the Crusader leaders who founded there several new states, a period known also as *Frankokratia.* The Principality of Achaia (or Morea) in the Peloponnese, led by Geoffrey I Villehardouin was the strongest and one of the most long-standing among them, lasting until 1430. To establish the new order and support the needs of the new settlers, the Villehardouin Princes created a few significant buildings in their newfound kingdom which included military infrastructure (like their stronghold in Chlemoutsi (*Claremont*) castle or the early phase of Mystras), but also several Catholic churches, abbeys, and other charitable foundations (e.g., hospitals). In an area dominated by the native Byzantine architecture, the patrons launched a construction programme that imported western designs and building techniques. Gothic elements and forms were thus applied in an alien domain to the movement, conditioned by local masons and material resources (Ousterhout 2010, p 262).

Although this period has been seen traditionally in Greece as a foreign, hostile occupation, recent research has demonstrated the existence of cross-cultural relations and exchanges in various aspects of everyday life, art, architecture, and even religious practices (e.g., Gerstel 2001, p 263; Mersch 2015, pp 462-3 and 466-7). The study of possible influences and exchanges through not only architectural forms but also the construction technology and structural design of these buildings can contribute to a deeper

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**Cite this article**

*Theodossopoulos D & Veloudaki C (2021). The Transmission of the Technology for Gothic Masonry Vaulting to Greece, in the Case of Saint Sophia in Andravida, Elis.* Cultural Heritage and Science, 2(2), 70-79

**Research Article**

**Received: 15/11/2021; Accepted: 17/12/2021**

understanding of the new conditions in the Frankish Morea, in particular technology developments.

As part of a wider ongoing project which examines the architectural technology landscape of the Principality, this paper assesses these issues through analysis of stone vaulting in churches, usually a key driver of the structural and spatial layout in Gothic buildings of the time. Churches may have been the most controversial field of the Frankish architecture due to dogmatic and cultural differences between the Roman and the Eastern churches and therefore this allows the exchanges between the two cultures to be more clearly tested.

The most prominent Frankish church surviving today is Saint Sophia in Andravida, the *Andreville* capital of the Principality. St Sophia is currently the only extant monument of the medieval town and offers therefore a unique example of how these new architectural schemes and technology were transplanted into a politically and culturally foreign environment. Although the building is only partially preserved, the vaults at the apse are almost intact (Fig. 1). Using them as a case study, the aim of this work is to examine possible cultural and technological exchanges as they can be identified through the vaults' architecture, construction process and structural behaviour. The performance of their whole system will be analysed by structural assessment of the vaults, as also assumptions about the design process.

The study also explores how a foreign design was imported by the new patrons and their agents the Mendicant (beggers) monastic orders, and the degree it was conditioned by the collaboration with local masons and the use of local material resources (Ousterhout 2008). This study can therefore provides more critical insights into design exchanges in Medieval Europe, bringing a different dimension to the mobility of central European workshops (e.g. German ones in 15th-century Castille, French ones in Angevin Naples in the 13th century etc).

Figure 1. The vaults over the apse in Hagia Sophia, Andravida

## 2. METHOD and CONTEXT

The main buildings of the Principality have been studied stylistically and historically (e.g., Athanasoulis 2013; Olympios and Schabel 2020, Campbell 2018), framed within meticulous earlier outlines of the history of the state (Bon 1969) or the spiritual and cultural appearance of the Mendicant monastic orders (Kitsiki 1979), but less has been written on their design and construction or the associated building culture. Grossman (2004) has a thought-provoking view of the period as an encounter between Western and Byzantine practices that lead to a new hybrid society and architecture. These buildings constitute Gothic architecture's most persistent expression in Greece and a brief reading of Byzantine architecture before the Frankish period is needed to map any possible technical exchanges.

### 2.1 Contemporary technical trends in Gothic and Byzantine architecture

The contact between the two cultures in the Morea was not peaceful initially but came in a period of technical refinements for both. The construction of Gothic cathedrals in France had reached large scales, like Chartres (1194-1264) or Amiens (1220-1266), through the continuous optimisation of structural schemes in stone like vaulting and vertical load-bearing elements (wall elevation, engaged piers). Ribs became important in the geometric resolution and materialisation of the difficult joints at the intersecting webs and were used eventually, together with the shafts from the piers, to visually unify the entire structure (*concordance*). Furthermore, the fusion of local styles into new universal Gothic forms were accompanied by the attitudes of the new mendicant orders for visual simplicity (Frankl 1962, p 150). These advancements permeated all scales of church-building at the time, so it is interesting to investigate whether they were transferred in the Morea.

Mainland Greece at the end of the Middle Byzantine period (843-1204) had gained once again an important role within the Empire (Mango 1978). Church architecture then was of a relatively small and intimate scale, extensively expressed in parish centres, which aimed to be perfect paradigms of the liturgical aspects of the Orthodox Church (Mango 1978; Ousterhout 1999).

The design trends of Byzantine churches that matter in this encounter focus around their interior space which is reflected in the strongly plastic treatment of the exterior (Mango 1978). The typical Byzantine hierarchy of the central dome and other elements shedding loads towards the vertical enclosure is extensively articulated in the smaller churches of the area.

Differences in the rituals can be seen at the emphasis on processional axes focusing on the single point, the altar, in the West in contrast to the centralised routes of the Byzantine church, even if basilican (axial) layouts were also without reaching though the gradual disintegration of the envelope as in Gothic architecture. Hierarchy in the construction plan was therefore important in both worlds but expressed with different means.

## 2.2 Frankish Architecture in the Morea

The Principality of Achax`ia (1205-1430) was led by the Villehardouins until 1278, when power was taken by the Angevins of Naples. Andravida was chosen as the capital and as is attested in the *Chronicle of Morea*[1], the official "narrative" of their state, the Princes built there their Palace, the church and hospital of St James, the church of St Stephen and that of St Sophia, where also civic functions like the Great Court took place. Although mentioned in various sources up until the early 20th century (Lambros 1916, 480-1; Lambros 1923, 101-3; Miller 1908, 97, 146), two of the churches, assumed to be St James and St Stephen, have little confirmed traces.

Other major new churches include St. Francis in Glarentza (*Clarentia* or *Clarence*) where the Principality's capital moved in the 1250s, which had vaults at the apse only, and several monastic churches that became important as the Principality was asserting itself, e.g. the Cistercian monastery at Zaraká (*Saracez*) in Stymphalia (ca. 1226-ca. 1263), its sister abbey of Notre Dame at Isova (ca. 1211-1263), and the church of St. Nicholas erected after the latter was burnt down (Olympios and Schabel 2020, p 165).

These Frankish churches were larger than their Byzantine contemporaries and had longitudinal basilican layouts, with no tripartite arrangement in the internal elevation, apart from some pointed arches at the windows. The ruinous state of the buildings today provides fragmentary information on the design of stone vaulting. There was probably no concordance of ribs with the shafts and St Sophia's intact vaulted choir has its ribs springing from corbels at their impost. As they are the only surviving genuine ribbed vaults, their technology is worthy to compare at a later stage of this project with the un-ribbed barrel vaults at Chlemoutsi.

The churches were commissioned by the new rulers for religious and possibly propaganda purposes and St Sophia may have been the flagship of the Princes in an urban context. This architecture imported by the new, foreign ruling class met local design attitudes that were less formulaic about styles, with more emphasis on the iconographic programme. Study therefore of the balance between patronage and building culture offers some interesting dimensions. In their construction, local materials like limestone were used but primarily brick was employed for the solid shell and there is a conservative exploration of structural efficiency and load-bearing capacity for vertical and shell-type loads in this new form.

## 2.3 The Church of St Sophia and its Dating Issues

The most intact Latin church of the Principality and at the same time a very enigmatic monument is St Sophia. As mentioned, it is the sole remain of *Andreville*, the first capital of the Villehardouins. Surrounded tightly by the modern town, only its east end is still standing today consisting of a sanctuary flanked by two side-chapels, all covered by ribbed cross vaults. The nave and side aisles are preserved only at foundation level, while the west end of the church is currently buried under a modern asphalt road (Fig. 2).[2] Despite its better state of preservation and the fact that the church has been studied by many scholars since the early 20th century (e.g., Traquair 1923; Bon 1969; Kitsiki 1979; Sheppard 1985 and 1986) its foundation date, history, and later uses remain obscure[3].

Contrary to the rest of the Frankish churches of the area, there are barely any mentions of St Sophia in literary sources, while sometimes these scarce pieces of information seem to contradict each another: St Sophia is assumed to have been the cathedral of the Bishop of Olena at least since 1205 (Rodd 1907, 174; Traquair 1923, 73) and also the

[1] The *Chronicle of the Morea* arrived to us in three versions: Greek, French, and Catalan, while there is also a summary of the Greek one in Italian. All three relate the history of the Principality, but they cover slightly different periods and do not always agree in detail. Occasionally one of the versions adds particular details lacking in the others and *vice versa*.

[2] The outline of the church could still be traced in the late 19th and early 20th century, see Christianike Archailogike Hetaireia, Deltion A' 1892, 98; Rodd 1907, 174; Traquair 1923, 73.

[3] According to Traquair ruins of a minaret to the NW of the church and remains of a mihrab in the interior indicate its use as a mosque during the Ottoman era (Traquair 1923, 73). Although this assumption is supported by Bon (Bon 1962, 557) and Kitsiki (Kitsiki 1979, 70), Sheppard doubts it (Sheppard 1985, 208).

court chapel of the Villehardouins with mentions of its religious and civic uses found in the *Chronicle of Morea* (e.g. Schmitt 1904, 380, 481). Yet according to the *Chronicle*'s Aragonese version, the only one making an explicit reference to the church origins, it was founded after 1264 by prince William II as a Dominican establishment (Morel-Fatio 1885, 77). Given that all references are rather imprecise, besides the local tradition, there is no absolute evidence to identify the surviving ruins with the cathedral (if there was one) or the church of the Dominicans[4]. The small-scale excavations carried out over the years either by the local Ephorate or others (The Minnesota-Andravida Project (MAP), see Cooper 1996; Sheppard, 1985 and 1986) have not yielded any conclusive results about the identity or dating of the building. For convenience therefore, the church will still be referred to simply as St. Sophia.

Especially the latter is a rather controversial subject as different scholars suggest slightly different time periods: E.g., Traquair who was the first to study and survey St Sophia in some detail, thinks that it was founded in the first quarter of the 13th century by Geoffrey de Villehardouin I (1210-1228/30), but the part which survives today he dates to the early 14th century built either by William de Villehardouin II (1246-78) or during the first years of the rule of the Anjou (1278-1376) (Traquair 1923, 76). Sheppard agrees with Traquair on the founding date but he is of the opinion that the existing vaults were added under the rule of William II before his imprisonment in 1259 (Sheppard 1985, 211). Bon believes it was built shortly before 1250 after the Dominicans' settlement in the Peloponnese (Bon 1969, 547), Kitsiki-Panagopoulos places its founding sometime after 1240, in the early years of the reign of William II (Kitsiki 1979, 66, 77), while Lock thinks it was between 1228 and 1264 (Lock 2013, 217, 232).

With the evidence currently at hand a firm conclusion cannot be reached (dates converge around 1240-59), nevertheless the dating of the church and the identification of its different construction phases is important not only for placing it in a wider historical and cultural context but also for interpreting its form and structure (i.e., Mendicant orders had certain architectural habits in accordance with their customs and apostolic missions and their buildings had to follow specific construction rules, so the Dominican churches reflect the order's limited resources and insistence on austerity and simplicity, see Coulson 1996; Kitsiki-Panagopoulos 1979, 65).

### 2.4 Method and the Research Project

The exploration of the technological history of the church will therefore require four research directions: historical-critical analysis; technology transfer; construction and style; structural analysis. This paper covers the last two areas primarily using bibliography, published surveys and a photographic overview from the authors (in 2008). The vaults will be analysed with Finite Element software and since they are the major structural element of the church, their behaviour inevitably conditions the stability of all the remains of the choir.

## 3. ARCHITECTURE and CONSTRUCTION

The work by Sheppard and subsequent publications appear as the most extensive and direct surveys of the monument, and some of their hypotheses on the history of the fabric are explored in this work or are suggested for future research.

St Sophia was a three-aisled basilica with no obvious transept and a square sanctuary of two bays flanked by two side chapels. Although it is not preserved in its full length today, from earlier surveys we know that it was about 45.50 m long by 18.85 m wide (Fig. 2), a substantial building[5]. The archaeology has showed that the nave had an arcade along the side aisles carried on rather slender single shaft columns, four of which lie still *in situ*[6], while the walls ranged in width between 0.90 m (N chapel) and 1.20 m (S chapel)(Cooper 1996, 32)[7]. The church was built mainly with local coursed poros stones (Kakouris 1979, 156), but red sandstone blocks, marble spolia as well as uncoursed rubble were used at certain places. Bricks and tiles were occasionally used as pinnings to level the courses and in the construction of later openings, so the masonry can be considered to be bonded well for structural purposes. The quadripartite ribbed cross vaults over the choir and the chapels are very well-preserved today. On the contrary, the nave and the side aisled were most likely covered with wooden trusses, with no trace of vaults springing or arcade bonding on the choir (Sheppard 1986, 142), meaning the choir can be studied as an independent structure.

[4] For example, the early 19th-century traveller F. Pouqueville who saw all three Frankish churches of the town standing, does not identify the bishop's church with St. Sophia (Pouqueville 1827, 367) and although the presence of Dominican foundations in the Morea in the early 14th century is documented, there is no evidence for Andravida itself (Coulson 1996, 50).

[5] The church was first measured by the Christian Archaeological Society in 1891 (Christianike Archailogike Hetaireia, Deltion A' 1892, 98) and although the later plans of Traquair and Bon give 53.70 m and 54.00 m respectively as the length, the more recent calculations of the MAP seem to corroborate the initial measurements (Cooper 1996, 29-30).

[6] In the area around the church there are parts of four blue-grey granite shafts, while until the late 19th century there were another four which were later moved to the cathedral of nearby Lechaina (Lamprou 1923, 102). The dimensions of both sets seem to match the height of the nave colonnades as indicated by the surviving impost blocks in the sanctuary's west wall (Cooper 1996, 33).

[7] According to Traquair the now buried western wall was considerably thicker than the rest (approx. 2.00 m) suggesting the importance given to the church's western front (Traquair 1923, 73), however, according to MAP's calculations it was no more than 1.00 m thick (Cooper 1996, 33).

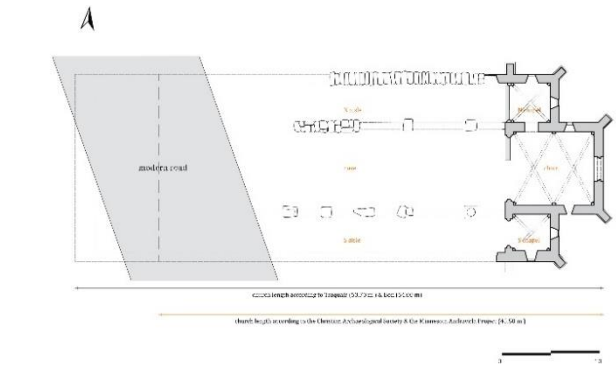

**Figure 2**. Plan of the church (after a combination of plans by Bon and Sheppard)

The high vaults are quadripartite, ribbed cross vaults, in a brick and stone course rubble (Fig. 3). The apse span towards the nave is 6.65 m and its width L is about 7.7 m wide, with the first bay being 4 m long and the second 4.4m. It was not possible to evaluate the overall height of the apse, but what matters here is the rise of the vault from its imposts to the keystone, which was assessed as F=5.73 m (Traquair 1923). The vaults geometric study shows the third point rule was followed for the design of the intersections and the ribs could have been built firstly to define the formwork. The resulting radius R is 5.9 m and was further used to trace the transverse arch of the apse towards the nave (triumphal arch). The arches along the lateral walls are then traced as projections of the diagonals on the wall plane.

On the side of the apse there are chapels that resemble the Byzantine pastophoria, with overall plan dimensions 3.95 x 3.65 m. Their ribbed vaults abut the high vaults by counteracting the line of thrust from the high vaults with their own thrust, assisted by the spandrels, the fill at haunches and the rather thick transverse walls (measuring between 0.94 and 1.02). The difference in rise between apse and chapels show the vaults were designed for this function of containing the thrust, which will be evaluated in the structural analysis of the vaults.

**Figure 3.** The ribbed vaults of the apse

The thrusts of the vaults fall furthermore inside the heavy pier buttresses built at the corners of the apse (Fig. 4) and only limited natural light gets in through pointed windows or lancets. The design of the vaults is plain, functional and well executed but it is not clear if extension of stone roofing to the nave was planned.

**Figure 4.** The diagonally laid pier buttresses at the exterior of the apse

Byzantine cloisonné brickwork bonds were used in the Morea (see the contemporary Byzantine churches in Glatsa, Merbaka, Manolada) and here they are attempted systematically only as decoration at the choir. On the external surface of the wall brick tiles were inserted probably during early restorations (Sheppard 1985), a point that needs clarification in future stages of this project. Overall, the masonry is coursed rubble that re-used ancient spolia and typically was meant to be visible. The vaults are formed of stone blocks, closely glued with thin mortar joints in a regular pattern (Fig. 3). It is clear an attention to the details in construction, reminiscent of the earlier vaults at the nearby Chlemoutsi Castle (1220-23). The Villehardouins showed they could invest in the quality of their buildings where needed and employ masons who could fulfil their aspirations.

As mentioned before, it would make sense to form the edge of the transverse vaults by projecting the ribs on the wall but apparently there was an effort to use for vaults and ribs the same radius, creating an awkward geometry which was not possible to materialise with the available expertise (Theodossopoulos 2009). The haunches had to follow this twisted pattern and hide behind the projection of the ribs and this peculiar detail was resolved by changing the coursing of the tiles to build a more solid cone (Fig. 5). All these techniques would not be visible as the vault was covered by plaster and there are traces of colour from creating a geometric pattern trying to simulate a stonework on the lateral walls. Some care of the detail in building the vault is quite visible, with careful adjustments and finishing of key elements, like the joining of the ribs on the corbelled support or the standardisation of the rib voussoirs as seen across all arches in the vaults.

**Figure 5.** The ribs and springings of the vaults

This overview shows an architecture that is consciously generated from the pointed arch, not merely making adjustments of the circular arch that was more in use in Byzantine vaults, indicating clear intentions by the patrons. From structural point of view, the haunches could have efficiently reduced the span required for the formwork and the layout of straight courses show no intentions for a domical geometry. However, the vaults do not represent the technical and aesthetic contemporary refinements of the type in Europe: the intersections along the groins are highlighted and strengthened by stone ribs of a heavy torus section (Fig. 5) and there is no attempt to unify the underlying space with ribs that extend as shafts to the elevation. Anyway, some authors, and among them Sheppard (1985) believes the ribs were attached later on the groins, at the time when the chapels were added, as the fallen rib on the north chapel indicates. This is crucial for the unity of the design and needs closer inspection in further research, to detect if the masonry along the intersections is disturbed or re-worked.

## 4. STRUCTURAL ANALYSIS

Analysis of the structural performance can assess the effectiveness of some of the design strategies and then address some of the historical assumptions mentioned earlier: whether ribs were integral to the shell (or used as permanent centering), the efficiency of buttresses (especially the diagonal arrangement), walls or aisle vaults in containing the thrusts. This may justify the construction sequence (aisles after the high vaults) and indicate the portion of the walls necessary for stability (and how wide openings could be made) and consequently if the high vaults were added safely later over pre-existing walls.

A Finite Element (FE) numerical model was formed to analyse the structural behaviour of the vaults under the dead load of the structure. The vaults were simulated like shells with the program GiD (as pre-processor) and Abaqus for the FE analysis and appropriate elements were used for the webs (shell) and the ribs and arches (3D beams) (Theodossopoulos 2003).

This simulation is only an initial approach to assess how the form testifies the various hypotheses for their thrust containment. A uniform thickness of 250 mm was assumed and the masonry was considered as isotropic, using values for a thin brickwork bond (elasticity modulus set conservatively as 5 $kN/mm^2$ (IStructE 2005, table 1). Linear elastic analysis under self weight 20 $kN/m^3$ was performed. Instability due to outward spread of the supports, the action that usually marks failure in vaults (Theodossopoulos 2008), was not assessed in this stage.

For the crucial simulation of the supports, the spandrel walls were modelled as deep arches along the edge of the shell: the width of their cross section was equal to the thickness of the wall (0.6m) and their depth was chosen as 1m to account for the change in the stiffness due to the lancet and pointed windows. The substantial lateral constraint of the side chapels (Fig. 1) was simulated to apply up until 2/3 of the height of the shell.

Under these assumptions, the deflections (Fig. 6a) are overall small, which is mostly due to the continuous elastic analysis, the rigid continuous form of the thin shell and the lateral constraints. Only a detailed 3D survey of the vault could show the magnitude of deformations experienced in its history and therefore the quality of the structural scheme. Also, the squint pier buttresses at the corners were not modelled as a presence or a diagonal constraint, since the stiffness of the supports in the FE model minimises any other effect. The removal of the ribs (Fig. 6b) causes an increase by 25% of the deflections, but follows the same pattern, showing that the keystone of the westernmost vault is most vulnerable.

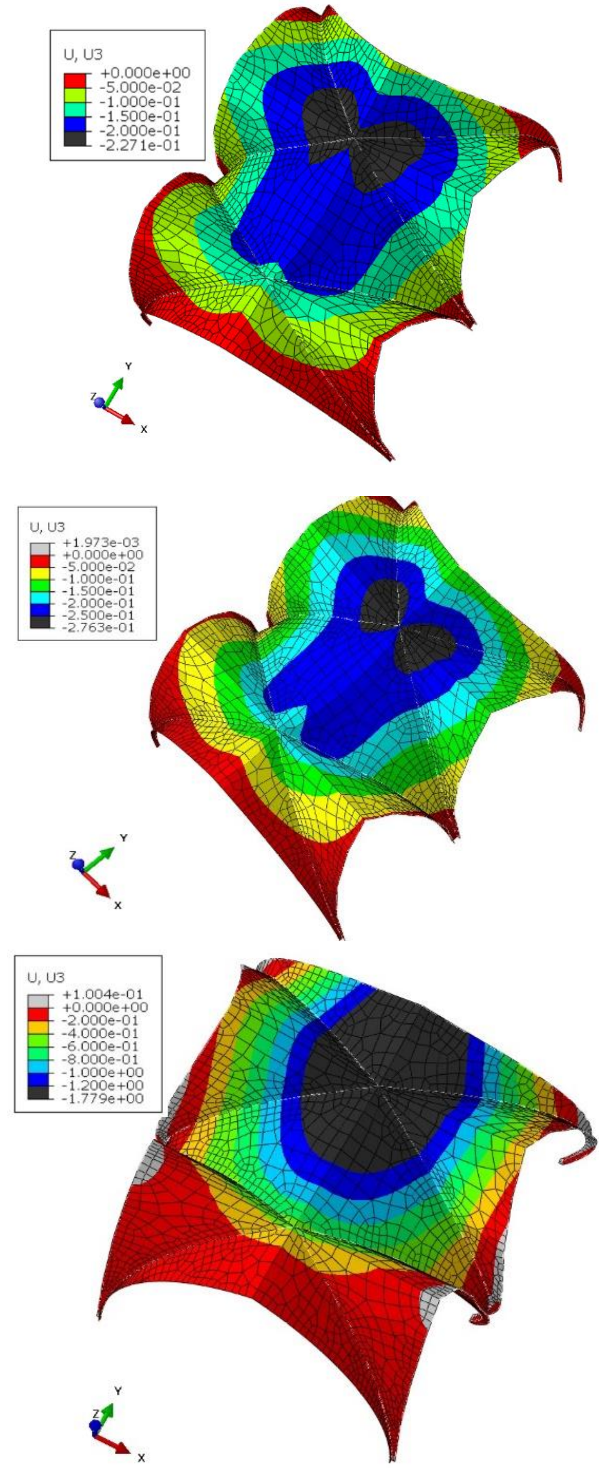


**Figure 6.** Deflections (in mm) from the analysis of the vault as a shell with ribs (a), without ribs (b) and with ribs but no lateral constraint (c)

Tensile transverse stresses appear along the longitudinal axis at the extrados (Fig. 7a), which show the positive effect of the lateral containment and the pointed form, which reverses the bending moment patterns in typical circular arches (tension at intrados of crown). Axial tensile stresses appear along the restrained wall edges at the intrados (Fig. 7b), which then spread towards the corresponding diagonal ribs.

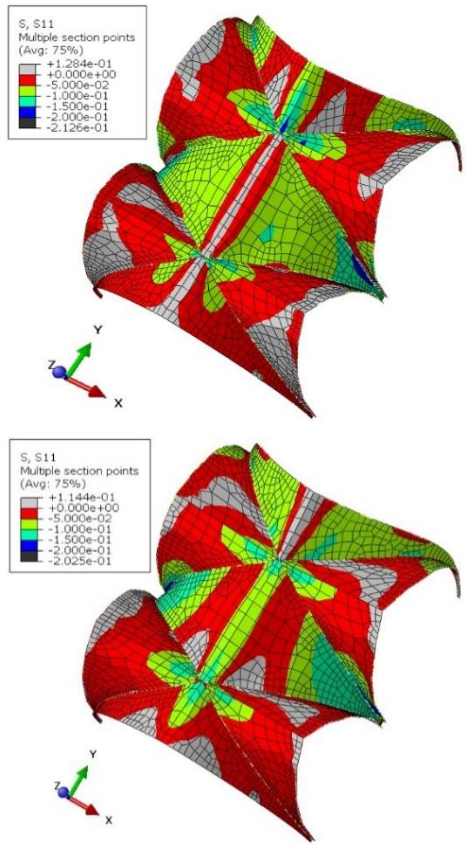


**Figure 7.** Transverse stresses S11 at the extrados (a) and intrados (b) of the vault with ribs, in N/mm$^2$

Finally, the effect of the lateral constraints of the chapels and their substantial roofs (Fig. 1) was removed altogether to test a hypothesis they were added later. This caused the vaults to spread differentially, outwards and inwards along the edge, while deflection increased strongly, extending to the west end and corresponding vault (Fig. 6c). This is a large deformation (within the conditions assumed) that would have left permanent effects any later additions of lateral constraints could not cancel. Survey of the vaults and any deviation from the ideal original design can once again clarify this hypothesis.

Seismic analysis should be explored in the next stages of this project, as the Elis area is historically vulnerable and recent earthquakes (2008, 2019) have apparently caused concerns, which resulted in the closure of the monument to the public.

## 5. REFLECTION ON DESIGN AND TECHNOLOGY TRANSFER

The analysis showed an overall successful and confident structural design for the vaults, which was probably carried out by an experienced, West-trained stone mason. Despite that, vaulting was not followed at the rest of the church and the timber roof that covered the nave was due probably not only to the restrained decorative attitudes of the Dominicans (if those were the founders) but also the lack of major architectural projects later by the Angevins, who after they took over from the Villehardouins in 1278 did not invest in the prosperity of Morea.

While the vault design could have been of direct Western import, the construction of the walls show the opportunities for collaborations with the expertise of the native masons. In most Byzantine churches of the time, the walls use the cloisonné pattern - framing squared stone blocks with bricks (Fig. 5). This type of bond, together with the tri-partite apse (which suits Eastern rites) and the use of brick masonry at the vaults come from the native building culture and can be seen in many churches in Elis. Heather Grossman's (2004) wider comparison of churches from both rites of the period that explored the case for a hybrid Moreote architecture serving both cultures, showed this architecture was complemented eventually by elements at the ornamentation, like the use of crocket capitals.

Vaulting is a more intensive area for technological transfer and it is believed that like Saint Sophia in Andravida, the churches of Zaraká (Campbell 2018) and St. Francis in Glarentza (Athanasoulis 2005) had such roof. In these cases, ribs seem they have been added along the intersection once the webs were built and thus did not have a function during construction (as can also be seen at the fallen ribs in the North chapel in Andravida and other repairs – see Sheppard 1985 p 206). In the native Byzantine practice, brickwork was used as a simple material for construction, with limited aesthetic treatment, and for adjustments of the form on-site. The joints however would be quite thick so mortar would set primarily because of the warm weather otherwise, in wetter conditions the buildings could easily distort. This is why any Western technology prototypes would be expected to have thinner joints and ashlar blocks.

Stone vaults apart from benefits in roofing, would be considered as extravagant elements, especially among Mendicant orders declaring and living the poverty like the Dominicans, who would restrict their use only at the apse (Kitsiki 1979) - if they were associated with Saint Sophia then the vaults are in line with this attitude. It was shows that even these plain vaults showed advanced technology and distinct articulated space qualities that could have been actively used by the patrons to promote the Principality and their Roman Catholic traditions to both the working and the upper native classes.

Flying buttress systems were not applied anywhere in Latin Greece. This is due to the relatively small scale of the churches and associated spans but also the lack of great ambition by the Princes which, in contrast to Lusignan Cyprus, did not have the conditions for producing such complex elements.

## 6. CONCLUSION

Gothic Architecture was a highly rationalised construction system that forms stone masonry into brave, efficient and slender stone skeleton-like schemes. This may have inspired the new patrons in Greece and the issues when transferring such schemes into the less technically efficient culture of the area at the time are explored in this work. The basic constructional and structural analysis conducted showed limited direct exchanges between the two cultures, which enabled though a successful implanting of Gothic ribbed vaulting in one of the major buildings of the period.

These vaults have a spatial concept that is quite far from the liturgical practices of the local Greek Orthodox architecture, where space is articulated by plastic treatment of the volume, and has no compositional elements like shafts and ribs. The apse however shows a common ground (Theodossopoulos 2009), the solid walls and vaults provide a controlled amount of light and thermal comfort. The cultural exchanges that may have taken place were highlighted by the Minnesota project (Sheppard 1985, 1986) which identified a later Frankish phase when openings at the apse were regularised and down-sized. This can show a need to integrate even contemporary Byzantine preferences for mystical and focused church spaces.

A measured survey of Saint Sophia can highlight deformations that may verify the structural design and clarify the construction phases. This can further demonstrate the processes of adaptations of original vaulting schemes and eventually the extent of cross-fertilisation between Western architectural intentions and local building practices. Any offsets should be distinguished carefully between adaptation efforts by inexperienced masons and structural problems by a weak design.

## ACKNOWLEDGMENTS

The authors want to thank Prof. R. Ousterhout for valuable insight to technology transfer and cultural exchanges during the Frankish period as also the Elis Archaeological Ephorate, and in particular the deputy director Dr. A. Ralli, for the permission to study the Frankish monuments of the region.

## Author contributions

Dimitris Theodossopoulos: Conceptualization, Methodology, Software, Writing-Original draft preparation, Validation
Christianna Veloudaki: Visualization, Investigation, Writing- Reviewing and Editing.

## Conflicts of interest

There is no conflict of interest between the authors.

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