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

**Srey Mom VUTH*[1], Paola PATERNA[1], Donato ABRUZZESE[1]**

[1]*University of Rome "Tor Vergata", Dept. of Civil Engineering & Computer Sciences, Rome, Italy*

**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

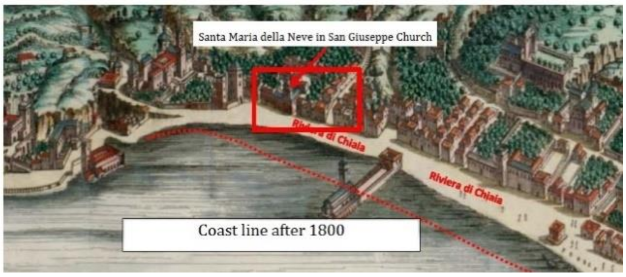


**Figure 2.** Map of Naples 1663 (detail)-from Stopendaal.

* **Corresponding Author**

*(sreymom.vuth@hotmail.com) ORCID ID 0000-0003-3402-7060
(paterna.paola@gmail.com) ORCID ID 0000-0001-5364-6700
(abruzzese@uniroma2.it) ORCID ID 0000-0003-0798-5239

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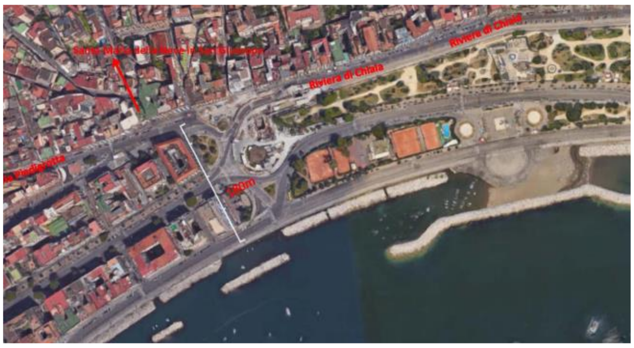

**Figure 3.** Republic Sq. and Riviera di Chiaia today (Google map in 2017)

## 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".

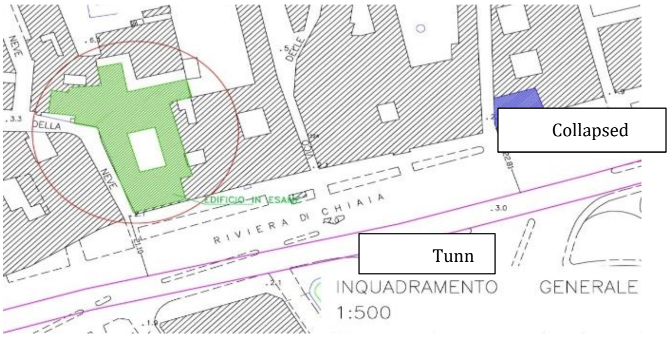


**Figure 9.** Map of the building, the tunnel line in construction and the collapsed building.

**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 |

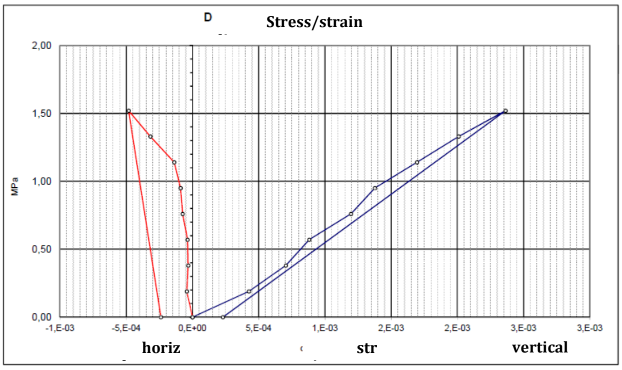


**Figure 12.** Test with flat jacks carried out on the ground floor wall

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.

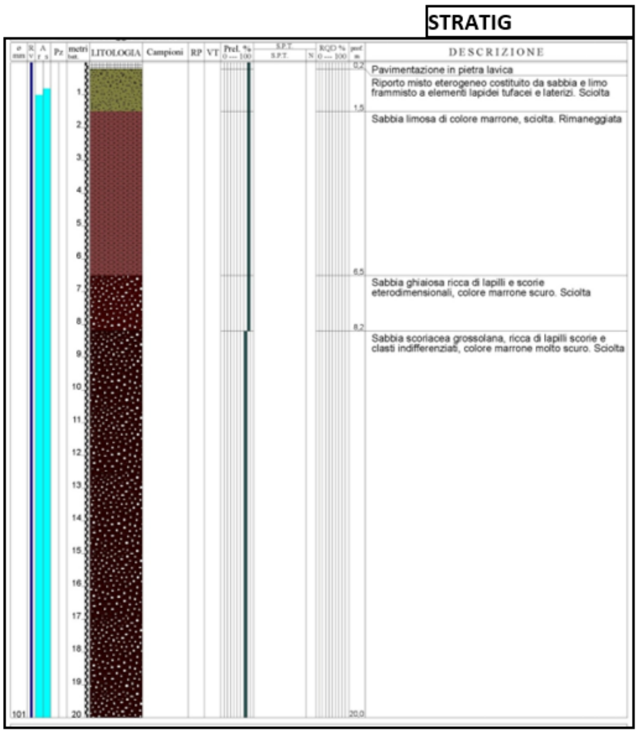


**Figure 14.** Stratigraphic column at the borehole S3

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”.

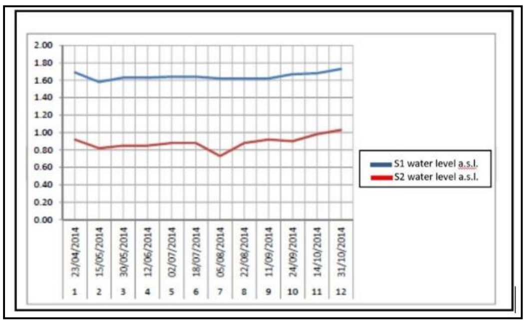


**Figure 15.** Piezometric control of the water table

#### 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.

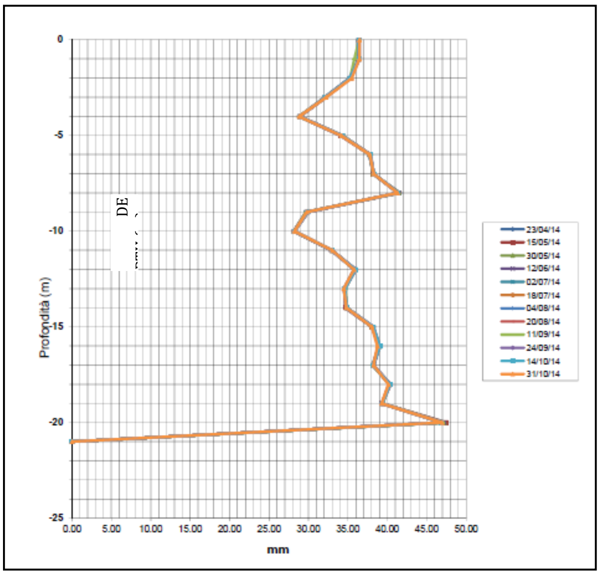


**Figure 16.** Horizontal displacement of soil borehole according to the inclinometer

#### 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.

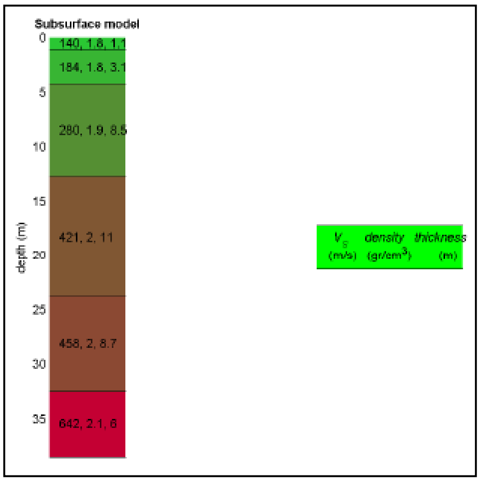


**Figure 17.** Soil density – one of the three MASW test

### 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.

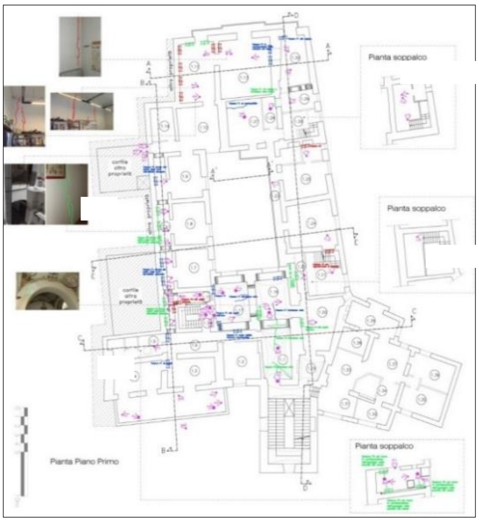


**Figure 18.** Cracking map at the first floor

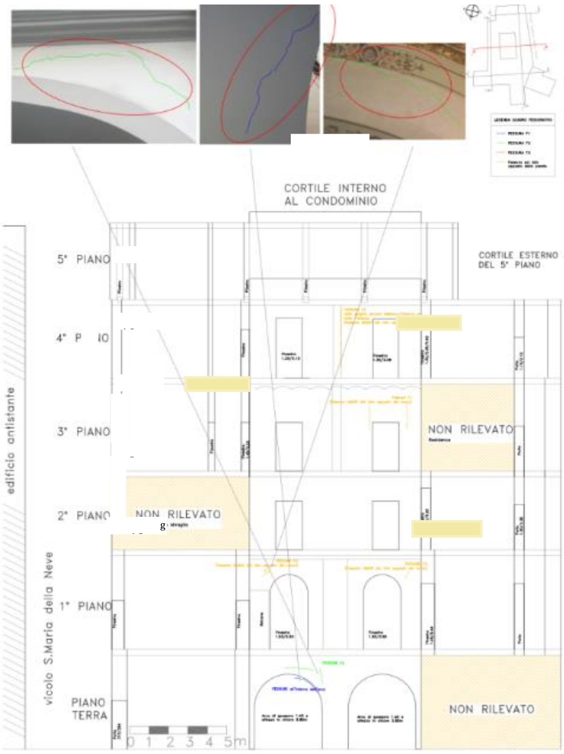


**Figure 19.** Crack map on the vertical walls in transversal cross-section E-E

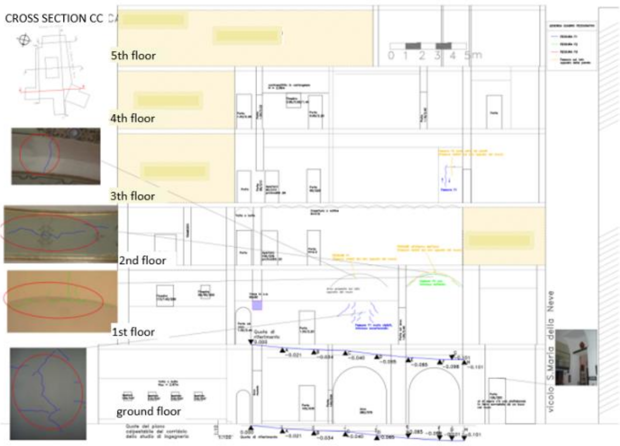


**Figure 20.** Crack map on the vertical wall in transversal cross-section C-C

**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:

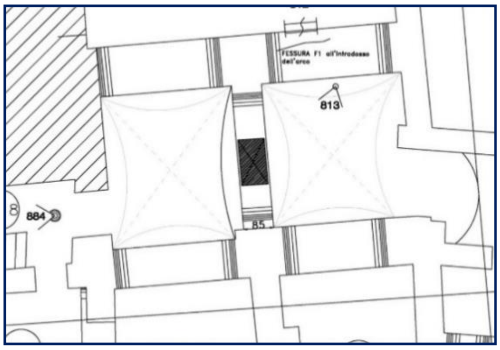


**Figure 23.** The plan of the cross vault

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".

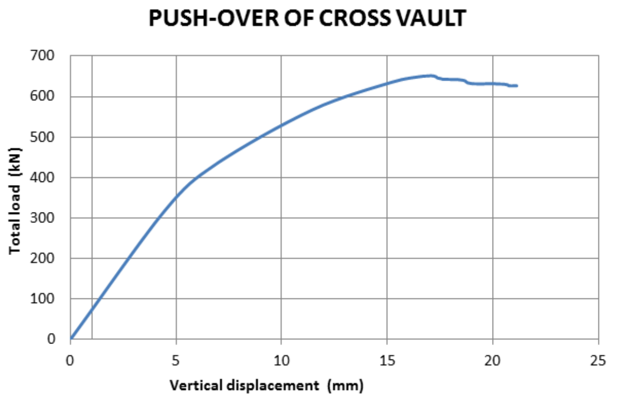


**Figure 24.** Result from vertical push over for cross vault

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:

$$\eta = 650\ \text{kN}/583\ \text{kN} = 1.12 > 1$$

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.

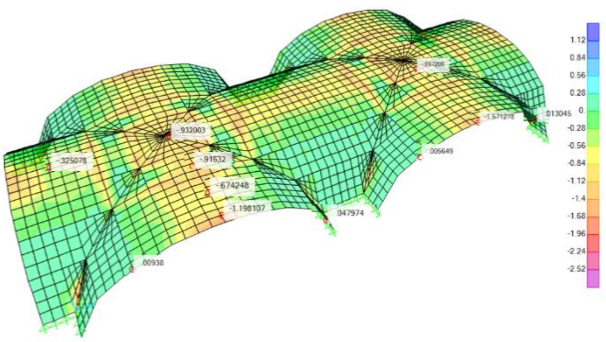

**Figure 25.** Tensile stress in the cross vault extrados

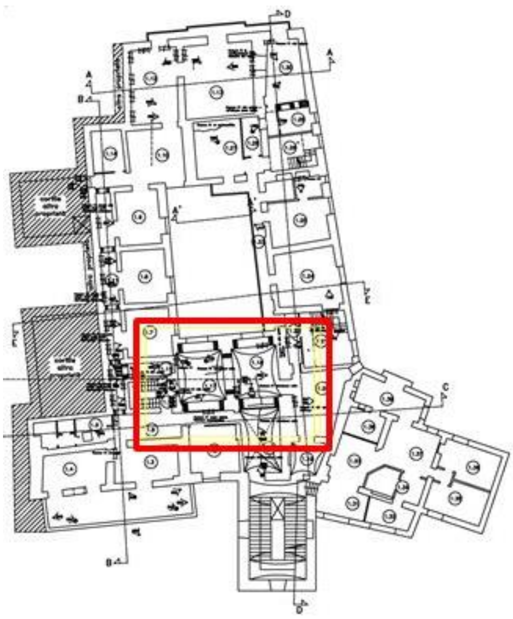

**Figure 26.** First-floor plan. In the red frame the two sail vaults.

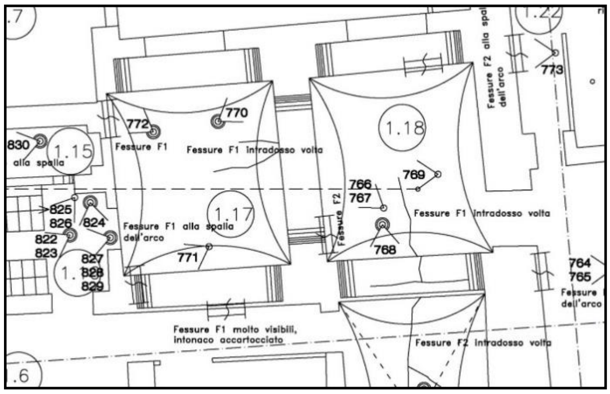

**Figure 27.** Plan of the sail vaults at the first floor

**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.

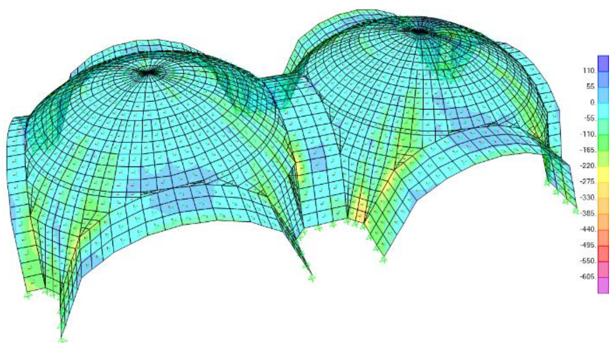

**Figure 29.** Mesh of the double sail vault and stress for ultimate live load

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

$$\eta = 1.078 > 1$$

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.

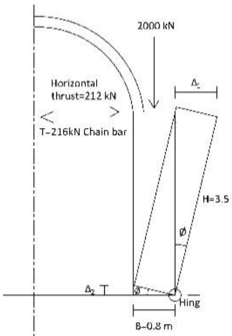


**Figure 30.** Equilibrium analysis for column, thrust, and steel chain bar for the sail vault.

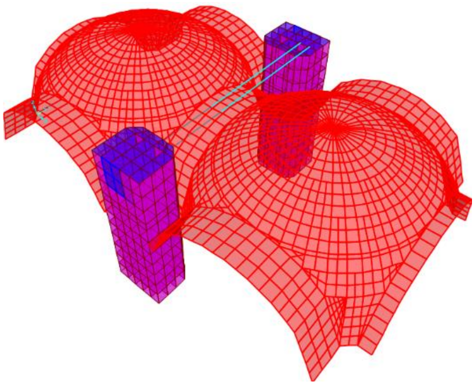

**Figure 31.** Graphic scheme of the two chain bars to be installed on the sail vaults.

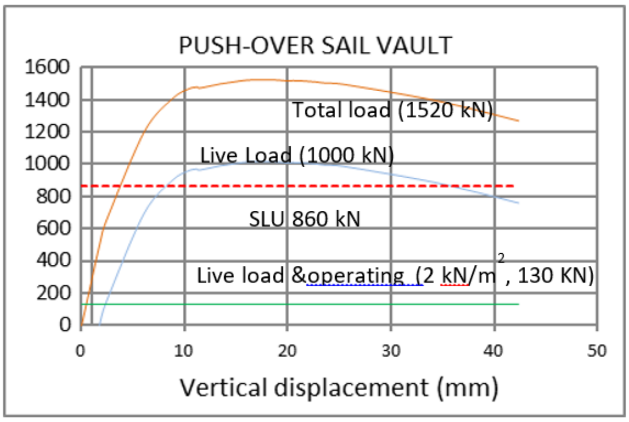


**Figure 32**. Sail vault Push-Over curves for different load conditions, without chain bar.

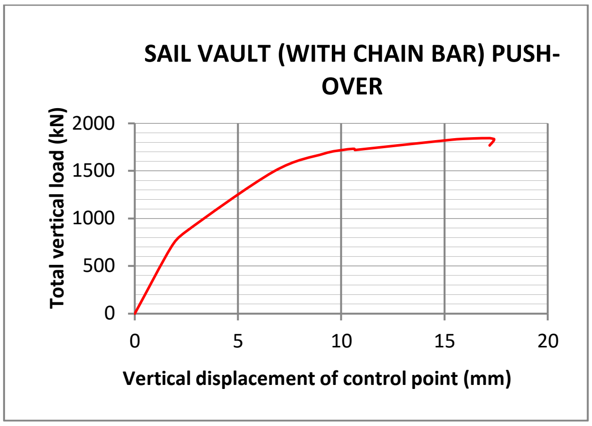


**Figure 33.** Pushover of the sail vault with the two chain bar. Total collapse load Qu= 1800 KN

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.

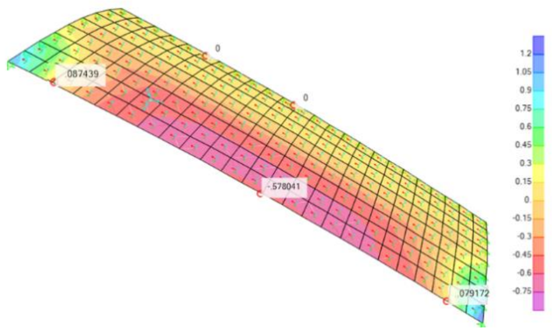

**Figure 35.** Stresses on the extrados of the stair vault due to ultimate load

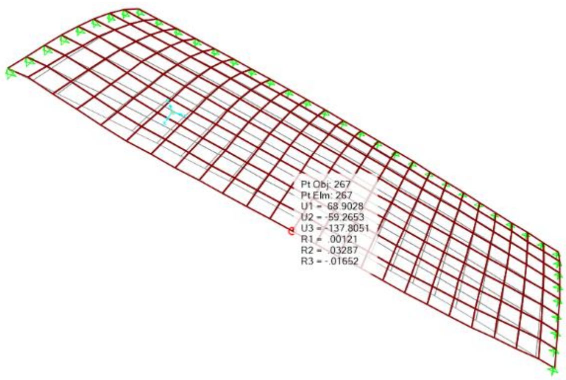

**Figure 36.** Displacement by ultimate load

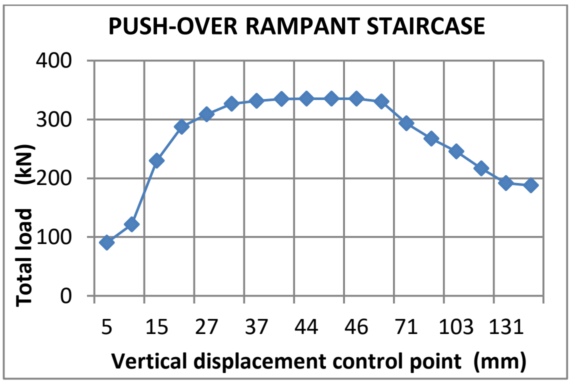


**Figure 37.** Pushover function for the load on the rampant staircase

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.

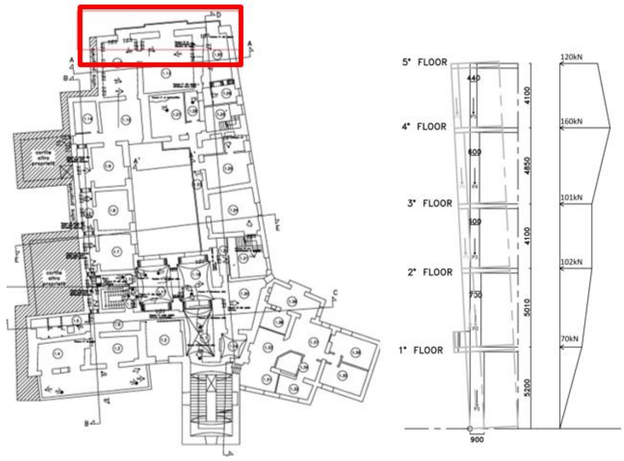


**Figure 38.** Overturning risk assessment of the main façade

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:

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

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

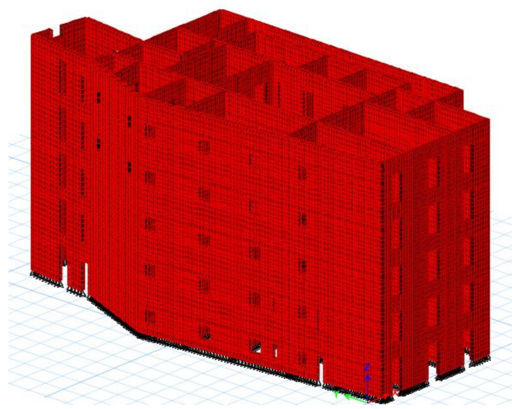

**Figure 39.** Global FEM model for the non-linear analysis of the building

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.

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.

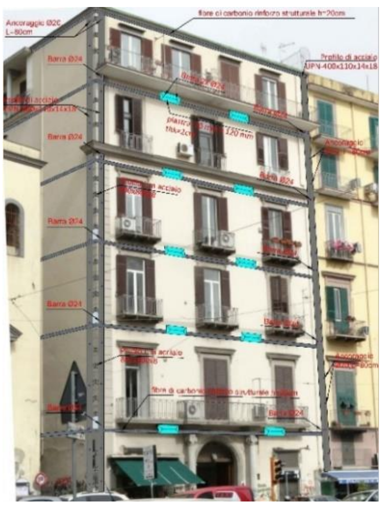

**Figure 40.** Rendering of some of the structural reinforcing on the external wall of the building

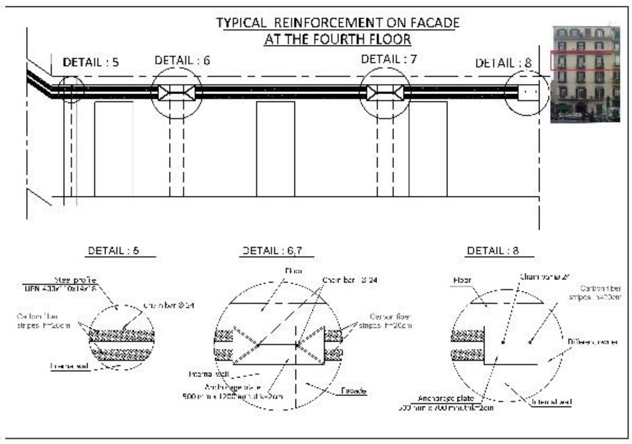


**Figure 41.** Details for the installation of the prestressed carbon fiber and the plates for the anchorage of the stainless chain bar.

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

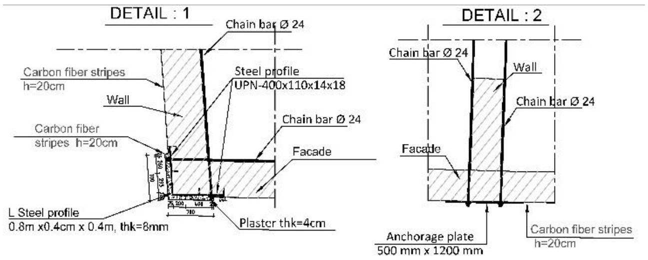


**Figure 42.** Detail of carbon fiber stripes and chain bar

**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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