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Numerical model and consolidation interventions of Palazzo della Ragione in Milan

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Abstract

Palazzo della Ragione, erected in 1233, represents one of the most ancient and relevant historic building of Milan. During the last century, the Palace suffered significant modifications, including the realization of an underground tunnel immediately near the foundations. Numerical analysis conducted with a FEM model were developed on the basis of some experimental tests. In particular, the diagnostic campaign performed in 1979, in which flat jacks and dynamic tests were applied, allowed to obtain useful information on the mechanical characterization of the masonry. In addition, the execution of some dynamic identification tests in 2017 returned the own frequencies of the building 40 years later. Before to work on the structural project, the autor verified the consistency between the structural response of the numerical model and the one of the real building, obtained by dynamic tests. Some consolidation interventions were realized on the wooden trusses of the cover, in order to restore either local and global safety situation, with respect to vertical and horizontal load.

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Keywords: Dynamic tests, Finite Element Model, Seismic strengthening

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1. Introduction

Palazzo della Ragione is the oldest masonry public building in Milan. The building comprises a lower part built in the 13th century and an upper part belonging to the 18th century, where poorer materials were used. During the last two centuries, until 1959, the building hosted the Notarial Archives of the town.

External dimensions are 18x50 m in plan and 28 m in height. The big room located at the first floor is covered by masonry vaults and it rests on isolated columns at the ground level. Wooden trusses, 18 m long, were originally adopted for the roof. In the 1960s and 70s, very severe load conditions developed on the building, not only due to the excavation of the first underground line close to the foundations, but also due to the repeated vibrations induced by the nearby trains. Thus, several cracks appeared on the main facades in addition to the ancient ones due to differential settlements. Due to these reasons, the municipality and many local newspapers were worried about the static condition of the historical building.

An authoritative proposal to remove the 18th century upper part was gaining ground in those years. Many architects and historians exspressed their opposition to this proposal and, at the end, it was decided to try to consolidate the whole building as it was. The aim of the initial investigations was to verify the real safety state and to propose interventions as minimal as possible within the concept of full respect for the building and its heritage.

Fig. 1. (a) Original configuration of Palazzo della Ragione in Milan (13th century); (b) Current configuration of Palazzo della Ragione

2. Diagnosis and consolidation interventions during the 1970s

2.1. In situ tests

The existence of a remarkable crack pattern affecting the external walls suggested an investigation of the soil properties to evaluate possible lack of homogeneity and, hence, different rates of settlement of the foundations.

Dynamic penetrometer and soil boring tests were performed near the building. Under the south-west corner, the most severely cracked, a looser soil was found which is probably due to the presence of an ancient channel crossing the area. A disturbed refilling soil with a high percentage of organic content was observed up to a depth of 12m. The column foundations reached the level of 8m, that is very deep compared with the original height of the building (18 m). This level probably coincided with the water table level at the time of the construction.

Mechanical characteristics of brick masonry walls and their in situ state of stress are usually difficult to obtain, especially when only a limited number of small size specimens can be extracted and tested.

To solve this problem the author applied a new testing technique, innovative fot that time, that is an extencion of the flat-jack test used in rock mechanic. This test was adopted for the first time in masonry. Useful results were obtained concerning in situ stresses at different locations, deformation parameters at different stress levels and limit strength values for the masonry. The obtained in situ vertical stresses varied between 0.2 and 1.2 MPa; the elastic modulus ranged from 3450 to 5100 Mpa.

Furthermore, some dynamic tests were performed on the masonry structures, to investigate their seismic response. Six accelerometers were positioned on the extrados of the vault to obtain the principal modes of vibration of the structure. Figure 2 shows few results obtained during the investigation campaign in 1980.

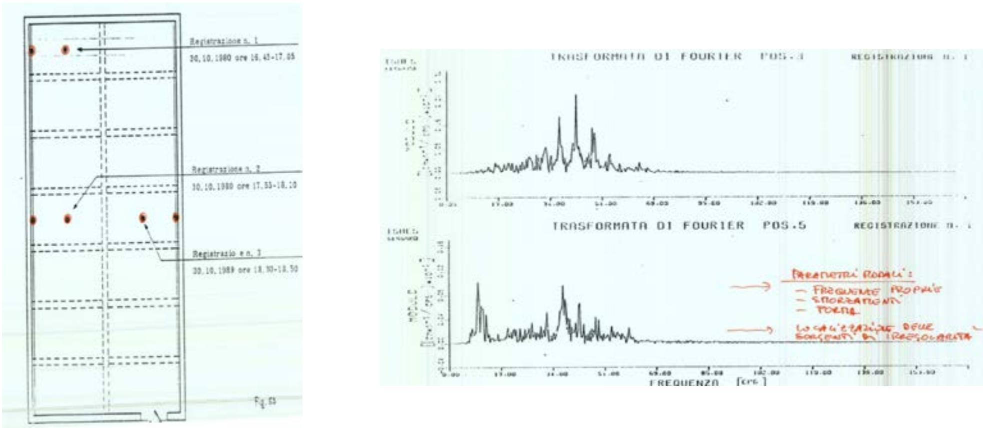

Fig. 2. Dynamic tests recorded on Palazzo della Ragione (1980). Plan view with the position of the accelerometers and accelerogram of two points after application of the Fast Fourier Transform (FFT)

2.2. Numerical Model

Serious cracks, apparently due to foundation settlement, could be observed in the masonry walls and an out-of-perpendicular displacement of about 25 cm was reached by the masonry walls near the top. In order to design structural reinforcements, an appropriate knowledge of the actual state of stress, mainly due to dead weight and to foundation settlements, was implemented. As the settlements were difficult to identify, even by means of accurate in situ and laboratory tests on the soil, an auxiliary and innovative procedure was proposed. Such procedure was based on the interpretation of the crack pattern present on the masonry walls, using a numerical model for the whole building.

An accurate survey and measure of the amplitude of the existing cracks was previously performed and a complete information about geometry, loads and constitutive laws of the materials was introduced into the finite element model (FEM) of the structure. Undamaged masonry was considered orthotropically elastic and the main cracks were modelled simply by disconnecting the nodal points. The possibility of reinforcing bars located at different positions were also introduced in the model.

Individual distorsions imposed at the base of each column cause relative displacements between the opposite sides of all the cracks, which were calculated and recorded. If combined and amplified properly, the effects of the different imposed settlement produce a distribution of openings of the cracks very similar to the actual and measured ones, minimizing the discrepancy .

It could be observed that the main damage was principally due to anomalous settlements of the corner columns and the ones adjacent, in two zones of the building.

Geotechnical tests in situ confirmed the particularly poor nature of the underlying soil, especially under the mentioned corner zones. In situ stress values measured by flat-jacks in 8 locations of the walls were also in agreement with the numerical analysis.

2.3. Consolidation interventions

The same finite element model adopted to determine the “true” settlements and, as a consequence, the actual state of stress was adopted to define the minimum reinforcement needed to increase the local safety of the walls in order to make it homogeneous throughout the building. The ratio between the radius of the limit Mohr circle and that of the concentric actual one was assumed to be the safety factor.

The numerical model allowed to study different patterns of internal and external reinforcing bars, in order to choose the most effective one. Mixed quartz-epoxy resin injections in the walls were simulated to locally improve the safety factor. Seven steel bars, 20 mm in diameter, were adopted for every arch, with a radial layout, thus obtaining a sort of continuous steel truss in between the arches and the lower windows.

Furthermore, a C-shaped continuous tie was adopted under the roofs wooden trusses. By introducing 14 pairs of X-crossed cables between the roof trusses, a semi-rigid diaphragm was obtained which strongly connected the four principal walls and made them to work together.

An external steel cable was installed at the level of the original roof, all around the perimeter, below the 18th century upper structure. This cable was fixed to the wall every 7 m and tensioned so that a confining effect in the masonry was obtained, where strong structural discontinuities were detected.

Reinforcing injections were used at the tensioned cables' reaction points, where the heads of 18th century chains were present, too. X-crossed bars were also used to reinforce the corners of the building, thus connecting the orthogonal walls. Using small diameter steel bars, the masonry vaults located below the first floor level were fixed to the principal walls, where they appeared to be disconnected.

No reinforcements were adopted for the foundations, as no more differential settlements were expected. Works ended with a soft cleaning of the 18th century masonry walls and with local nailing of the protective mortar plaster.

The temporary steel ties connecting all the columns on the ground floor since 1959 were finally removed.

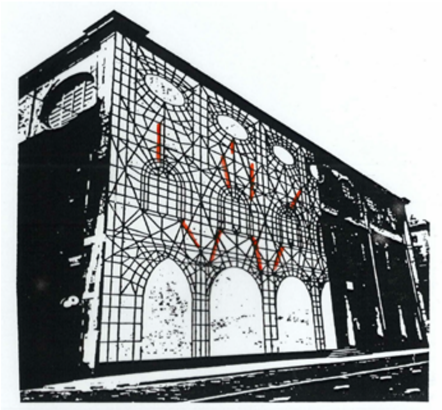


Fig. 3. (a) FE Model implemented in 1980s to determine the cause of the crack pattern through a “back analysis”; (b) “C” shape steel element and bracing structures introduced in 1980’s

3. Diagnosis and consolidation interventions in 2017

3.1. Relief, in situ tests and FE Modelling

During the last decades, Palazzo della Ragione was subjected to the natural decay of the masonry and wooden structures, so that the Municipality of Milan asked a new deep analysis to investigate the actual structural response in terms of vertical and horizontal loads.

Numerical analysis conducted with a FE model were developed on the basis of some experimental tests. In particular, the diagnostic campaign performed in 1979, in which flat jacks and dynamic tests were applied, allowed to obtain useful information on the mechanical characterization of the masonry. In addition, the execution of new dynamic identification tests in 2017 has returned the own frequencies of the building 40 years later.

These recent surveys, alongside with an accurate geometric relief by laser scanner, led to a more refined and updated FEM analysis than the one developed in the 1980s.

Before conducting the structural verifications, the consistency between the structural response of the numerical model and the one of the real building, obtained by dynamic tests, was carried out. Through these results, modal frequencies and the presence of "phase" or "out of phase" movements were determined at various points in the building, comparing the numerical data with the experimental one.

It was an iterative process of refining the numerical model, based on the reduction in the difference between the FEM and the experimental result, recorded in situ, in terms of periods of vibration.

The calibration process operated parametrically on the type of constraints and the elastic modulus values. Concerning the constraints at the base of the masonry walls, two alternative conditions were modeled, assuming perfectly hinged or clamped joints. The discrepancy with the average of the in situ measured frequencies was equal to 1.34% for hinged constraints and 14.5% for clamped constraints. Due to the minor difference obtained in the first case, the model with hinge constraints was used.

From the modal analysis, the following first frequencies of the structure were obtained:

Table 1 – First frequency obtained by the FE Model considering 2 different constrained conditions

	Hinge	Clamp
F1 [Hz]	1.358	1.534

Through on-site measurements the first frequencies of the building were detected:

Table 2 – First frequencies detected during the in-situ tests

	frequency	Modal type
F1 [Hz]	1.34	north-south flexural component
F2 [Hz]	2.43	north-south torsional component

To "calibrate" the first frequency of the FEM model so as to make it as close as possible to the in situ measured frequency, some variations on the elastic modulus E of the masonry were applied. Using an iterative procedure and calculating the M/R ratio (Mass/Stiffness) for various values of the elastic modulus, a "calibrated" elastic modulus of E = 1373 N/mm2 was obtained, with a difference from the module detected by flat jack in the order of 8.47%, that is an acceptable value.

By inserting this "calibrated" elastic modulus into the FEM model, a first frequency f1 = 1.35 Hz was obtained, which is a value not far from the experimental data. With this "calibrated" elastic module, stress and deformations of the global model were subsequently calculated.

In addition, the modal deformations obtained from the FEM model respect the phases and counter phases highlighted by on-site dynamic surveys, conducted to obtain the own frequencies of the building by measuring environmental microtremors.

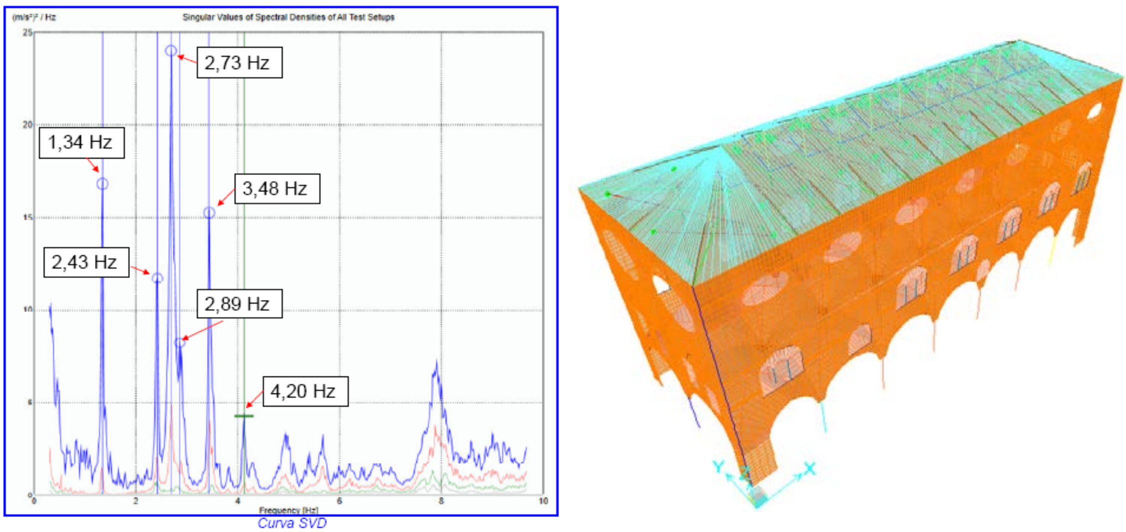

Fig. 4. (a) Frequencies obtained by dynamic test "; (b) FE Model implemented (2017)

3.2. Consolidation interventions

The consolidation interventions regarded mainly the roof.

First, a passivating treatment was applied on the perimeter "C" shape steel element that was introduced in the 80s. The original bracing structures were replaced by new stainless steel cables, positioned horizontally at the level of the chain of the wooden trusses.

Then, wooden trusses were strengthened through an innovative solution that consists in an auxiliary structure, composed by two wooden props, located approximately at mid-span of the struts, and four steel diagonal bars, in order to support these props from below.

For each half of the wooden truss, this system creates a sort of triangular pattern that includes the diagonal strut, the existing wooden chain, the new wooden props and the couple of stainless steel bars.

The prop works as an intermediate support to the strut, whose inflection is strongly reduced. At the same time the prop, that work in compression, would transfer a vertical force to the wooden chain, that, as a consequence, would became subjected to an undesired bending moment.

The couple of bars are connected to the wooden chain in a position under the new prop and, once tensioned by nuts, a vertical force, directed from the bottom to the top, is transferred to the chain under the prop. This vertical force eliminates the bending moment in the chain, which can continue in its original function, working only in traction.

The system is simple, resilient and active and can be installed without any need to dismantle the roof, that is a particularly useful feature both for costs and for times of the building operations.

In order to guarantee a box-like behavior of the entire upper part of the building, especially in case of seismic events, the two supports of each wooden trusses were connected to the perimeter "C" steel profile through an "inverted V" shaped element, made by a thin strip of steel. In this way the wooden chains, mutually linked to the perimeter walls, are able to work as struts and ties for horizontal loads.

Furthermore, to improve the global structural behavior in case of earthquake three layers of cross wooden planks were positioned on the roof. All the planks are connected each other and to the below wooden trusses by nails, so as to to create an useful semi-rigid diaphragm.

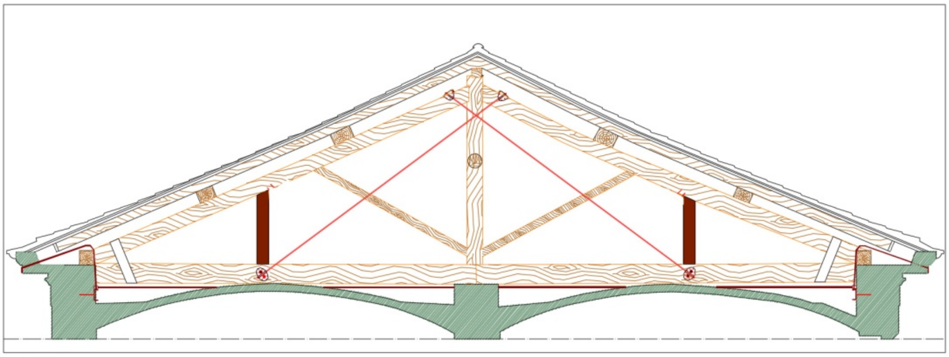

Fig. 5. (a) Frequencies obtained by dynamic test "; (b) FE Model implemented (2017)

3.3. A further proposal for the seismic retrofit

The relief of the deformation and the crack pattern of the building confirmed the significant phenomenon of out-of-perpendicular displacement of the longitudinal walls at the level of the string course connecting the elevated eighteenth century and the original building, belonging to the thirteenth century.

The out of plumb phenomenon was also included in the FEM model, faithfully reproducing the current geometry of the building, aiming to examine the dynamic behavior of this wall, analyzing the stress state and verifing the effects of the consolidation interventions.

Thus, an additional project proposal was introduced, which consisted in the insertion of a system of steel cables, placed at the level of the string course, which acts as a bracing of the building at an intermediate level between the first floor and the covering.

Three different situation were considered in computations:

1)The unconsolidated situation;

2)The current situation where the roof is consolidated, where the bracing contribution of the roof can be schematized with translational springs, i.e. with a yielding constraint;

3) The project situation, with the addition of a bracing plane. at an intermediate level of the big room

The main results in terms of displacements and stresses under seismic load are summarized in table 3.

Table 3 – Comparison between the unconsolidated situation, the actual situation and the project one

	(1) Unconsolidated situation	(2) Current situation	(3) Project situation
Max_Displacement [mm]	29	21	14
Max_tensile stress [MPa]	0.43	0.30	0.22
Tensile Area [%]	49	33	22

As expected, the proposed “braked shape” intermediate bracing structure offers a significant contribution to the global behaviour of Palazzo della Ragione, in presence of seismic events.

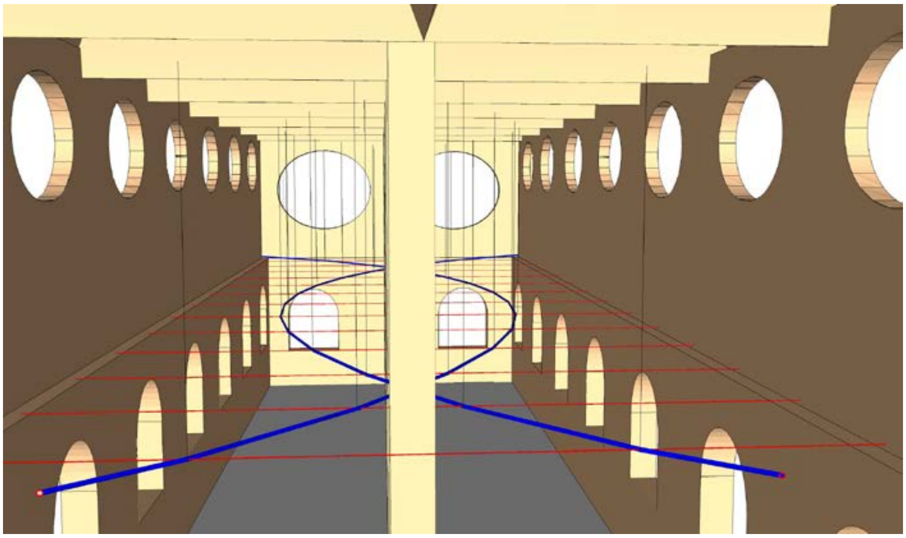

Fig. 6. Bracing of the building at an intermediate level of the big room.

4. Conclusions

Historic buildings require a special care during each design phase, starting from the preliminary analysis of the conservation conditions, passing through the diagniostic phase in order to project consolidation interventions that arre effective and respectfull of the building.

In this paper the special case of Palazzo della Ragione in Milan has been treated. A first diagnostic campaign was conducted by the author in 1970s, after which some consolidation interventions were realized.

In 2017, due to a structural decay, a more accurate diagnostic campaign was conducted in terms of geometric relief and dynamic tests, that allow to calibrate a new FE Model.

Some consolidation intervention were designed and realized, mainly at the roof level, to improve the global behaviour of the structure under vertical loads and in case of seismic events.

A further bracing structure was proposed, but not jet realized, to improve even more the structural response of Palazzo della Ragione. Relevant reductions of maximum stresses, percentage of tensile area and displacements were numerically detected.

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