Dynamic Characterisation and Seismic Vulnerability Assessment of Existing Masonry Port Structures

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1 Introduction

In recent years, the masonry construction sector has shown a growing vitality, both from a cultural point of view and from that of the building activity. This recovery has certainly contributed to the pressing interest on the part of the scientific community in the rehabilitation, restoration and seismic retrofitting of the enormous heritage built in Italy.

It was therefore taken note of a significant problem that afflicts our country, the preservation and reuse of existing buildings. In any intervention concerning the building heritage it is necessary, in addition to the knowledge of the state of conservation of the masonry, to investigate the design criteria, the construction techniques and the rule of art adopted during the construction of the artifact. It is also of fundamental importance to define the mechanical behaviour of both recently built and historical masonries.

The modelling and seismic analysis of masonry buildings topic is therefore absolutely topical. In fact, the assessment of the seismic vulnerability of existing masonry structures represents one of the most critical aspects that the civil engineer and, specifically, the structural engineer must face. A valid support on which the professional can rely is represented by Structural Health Monitoring (SHM), a tool of fundamental importance that allows to observe and estimate the state of health of civil works for the purpose of assessing their safety.

One of the key steps of the seismic vulnerability assessment lies in the design of a structural model on which to carry out static and dynamic analyses. Often, due to the lack of information on the structure, the model created on the basis of historical-critical analysis and classical investigation techniques does not constitute a perfect digital twin of the real structure and, consequently, a sufficiently faithful representation of the real dynamic behaviour of the latter. In order to match the behaviour of the model with that of the real structure, the methodology of dynamic identification can be used. The purpose of the latter is to estimate the modal parameters of the building, on the basis of which it is possible to calibrate the finite element model.

In the next chapters the seismic vulnerability assessment of the former railway station "Livorno Porto Vecchio" will be analysed, starting from the historical-critical analysis, ending up to the numerical modelling, as well as the Structural Health Monitoring (SHM) and the dynamic identification. We believe that the present work might be a useful contribution for the enhancement of the scientific background regarding the Mediterranean and European built heritage.

2 General overview

Located in Piazza del Portuale n°4 (Calata Carrara), 57123, Livorno, the building under study is part of the building complex constituting the former Livorno Porto Vecchio railway station, until 1939 called Stazione Marittima, today a branch of the Port System Authority of the Northern Tyrrhenian Sea.

The building, almost symmetrical with respect to one of the main directions, can be virtually divided into 5 blocks (Fig. 1). Blocks 1,2,4 and 5 are divided into 2 levels, while block 3 is divided into a single level. The first floor slabs of blocks 1,2,4,5 are located at the same height (3,50m), while the roof slab of block 3 is located at a higher altitude (5,20m). Finally, the roofing floors of blocks 1,2,4,5 stand at a height of 7,50m. Between the buildings 1,2 and 4,5 there is a structural joint that will be hypothesized to be effective also from the seismic point of view (Fig. 1).

All blocks are regular in plan, while only blocks 1 and 5 are regular even in elevation. This will affect the calculation of the behaviour factor *q* (NTC2018).

The load-bearing structure consists of masonry walls of various thicknesses; the perimeter ones have a regular distribution of the openings in height. The horizons are one-way concrete slabs.

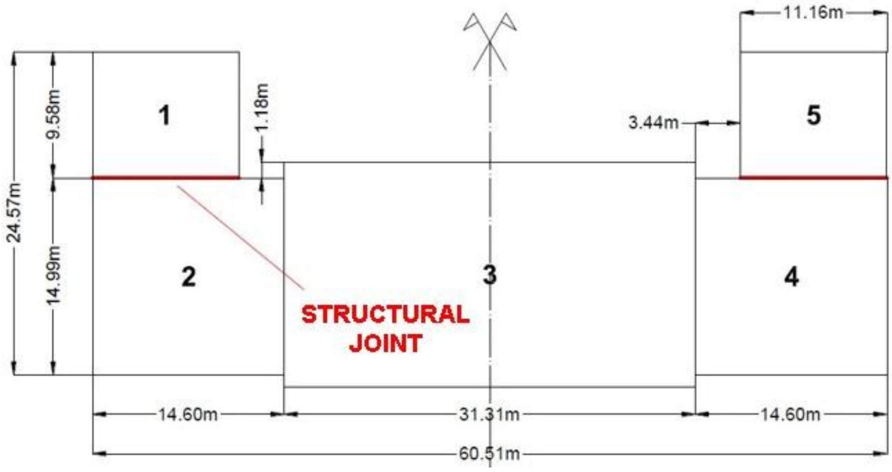

Fig. 1. Structure block schematisation.

3 Historical-critical analysis

The building in question is a former goods yard located in the port area of Livorno, now a branch of the Port System Authority of the Northern Tyrrhenian Sea, which allowed to connect the Port of Livorno to the railway. An important work built on land taken from the sea.

The works of the railway yard, intended exclusively for freight service, began in 1856 and were completed in 1858. A vast dock was created, flanked by railway tracks and sheds used for the storage of merchandise. In addition, a building for office use and a small building were erected for the Guardia di Finanza, the latter in the middle of the water.

The area suffered extensive damage during World War II (1939-1945). The main building of the station was destroyed by bombing and promptly rebuilt in the post-war period, although in different forms (Fig. 2).

Over the years, the traffic centre of gravity was moved to the docks of the New Port of Livorno. The station gradually lost importance and the new building and sheds were used for other uses; moreover, the dock facing the station was filled.

In the course of time, the structure has undergone changes in the distribution of the interiors, which have led to the demolition and reconstruction of some partitions.

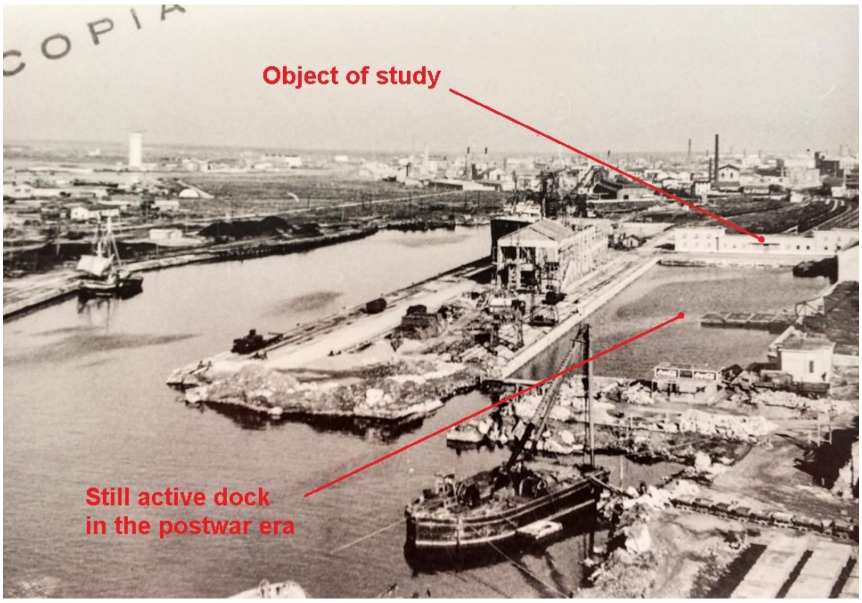

Fig. 2. Rebuilding after WWII.

The historical-critical analysis has also allowed us to detect the legislation in force at the time and the available existing documentation.

The following summarizes the main results of the historical-critical analysis and highlights the first critical issues and the main observations:

- Soil: There is no geological report referring to the area on which the building insists, but it was possible to find one relating to an area located about 300m away. According to what is reported inside, taking into account the geophysical surveys in a down hole type hole carried out, a soil category C was assigned (NTC2018);
- Geometry: from the existing documentation it was possible to obtain the thicknesses of the structural elements and the interstory heights;
- Slabs and loads: from the certificate of static suitability found it was possible to obtain the stratigraphy of the horizons and the related permanent loads.

No information was found regarding the foundation apparatus, the degree of interlocking of the walls, the presence of curbs and lintels, the warping of the floors, and so on.

In order to verify the correctness of the information obtained from the historical-critical analysis and obtain the missing information, a campaign of investigations and tests has been designed, as reported in the following paragraphs.

4 On-site testing and investigation campaign

In the preliminary phase, all the available documentation was analysed. This analysis highlighted the lack of useful information for accurate structural modelling. Therefore, it was necessary to provide for an extensive plan of inspections and tests.

In the first place, visual investigations were carried out that made it possible to examine the wall texture and its degree of interlocking, to analyse the cracking pattern and to evaluate the state of general damage to the structure. Based on these investigations, several tests, including non-destructive (thermography, sclerometer, cover meter) and minor-destructive ones (single and double flat jacks, shove test, penetrometer), were planned in order to assess the mechanical properties of the constituent materials, as shown in Fig. 3 [1-3].

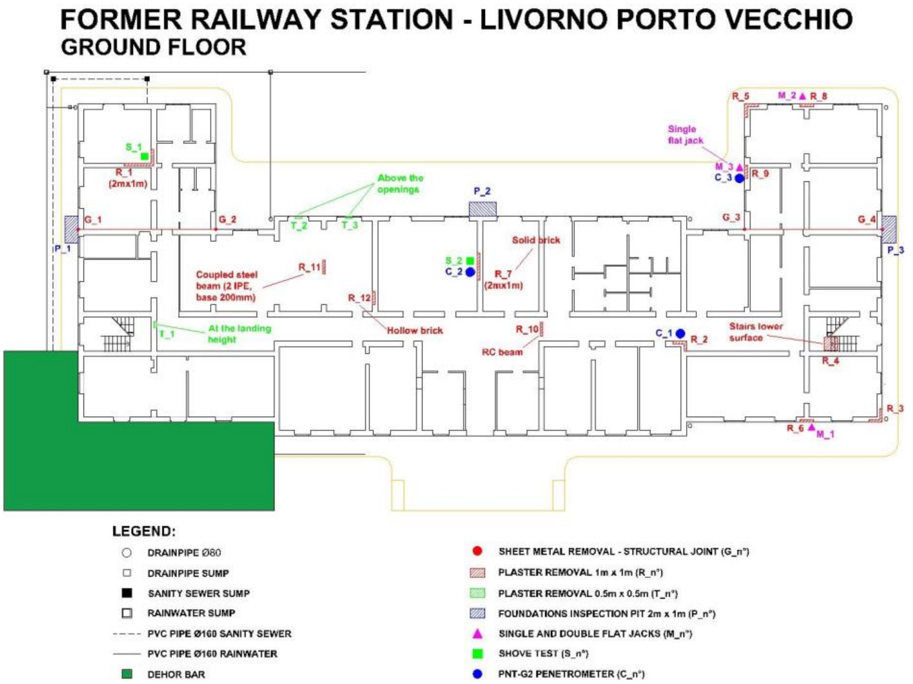

Fig. 3. Testing and investigation plan.

5 Dynamic identification

5.1 SHM acquisition system

The LUNITEK data acquisition and processing system model TRITON was used. The system is equipped with an integrated Force Balance triaxial accelerometric sensor. The 24-bit resolution A/D conversion card was programmed with a sampling rate of

250 samples/second per channel with synchronous sampling. The system is equipped with a microSD memory that allows you to manage a ring-buffer for long continuous recordings and a GPS receiver for synchronization in absolute time. The connection to the instrument can be ensured via local network connection (cable or WiFi) or via UMTS / HSPA modem that allows you to control its operation remotely. The system is powered by the external network and integrates an internal battery (LiPo) that guarantees continuous operation for over 30 hours. The system was used for multi-channel monitoring by installing multiple units [1-3].

5.2 Sensor's setup

All accelerometers in the various configurations have been oriented with respect to a common global reference system (N,E,Z). Fig. 4. Setup example.
shows a configuration example adopted during the dynamic characterization tests of the building.

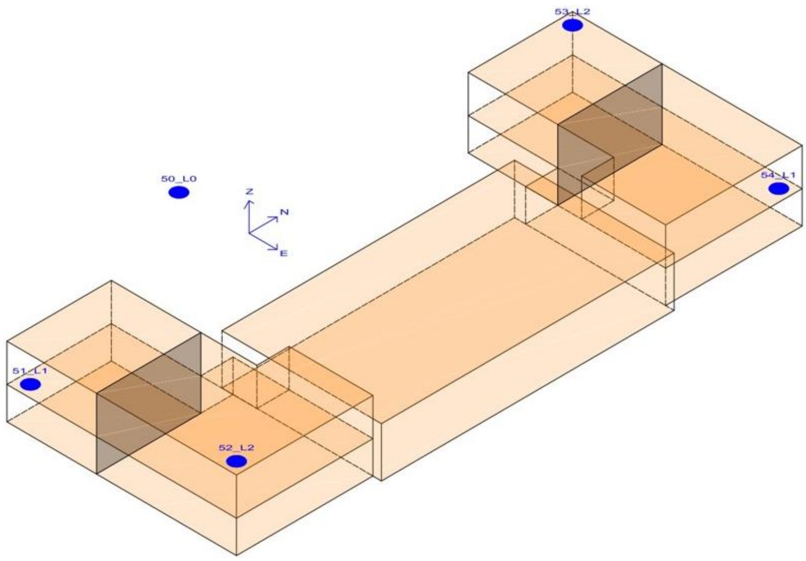

Fig. 4. Setup example.

5.3 Ambient Vibration Test (AVT)

According to operational modal analysis, the building was subjected to Ambient Vibration Tests (AVT), in order to determine its modal characteristics, such as natural frequencies and modal shapes [1-4].

The data acquired by the sensors are acceleration time histories, the actual duration of which was approximately 15min per single setup. Within this recording window, a "clean" sub-window was then identified, that is, free of anomalous peaks due for example to bumps, and therefore identifiable as white noise Fig. 5.

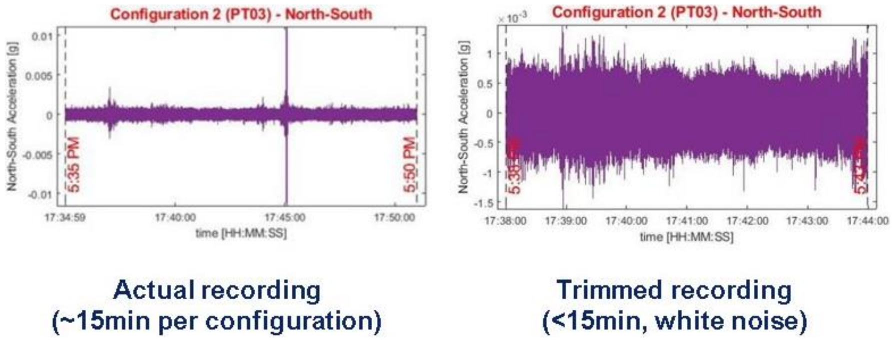

Fig. 5. Example of acceleration time history.

5.4 Filtering

By means of a MATLAB script, appropriate signal theory techniques were applied. In particular, we applied a low-pass filter, with a cut-off frequency equal to 30Hz, and a band-stop filter to reduce frequency peaks attributable to external noise sources (Fig. 6).

Fig. 6. Filtering.

5.5 Frequency Domain Decomposition (FDD)

Subsequently, we applied the dynamic identification technique called Frequency Domain Decomposition (FDD), based on the singular value decomposition algorithm of the Power Spectral Density (PSD) matrix.

The singular values are returned in output, from whose analysis it is possible to identify the spectral density eigenfunctions corresponding to each mode of the system and, therefore, to estimate the modal shape at each peak [1-4].

5.6 Results

In most of the results it is possible to observe a frequency response peak located around 3.5Hz, which could be attributed to the structure. However, on closer inspection, a serious mistake would be made.

Although not necessary in the case of dynamic output-only identification, a ground sensor was also installed during the tests. By analysing the PSDs of the signals from individual accelerometers, an unexpected discovery was made. The 3.5Hz tone is simultaneously present both for the accelerometers positioned on the structure and on the ground (Fig. 7), and in all 3 instrumental directions. This reasonably means that it is not referred to the structure but depends on the frequency component of an external input source. In light of what was discovered, it was not possible to identify the modal parameters of the structure and therefore to calibrate the subsequent structural model.

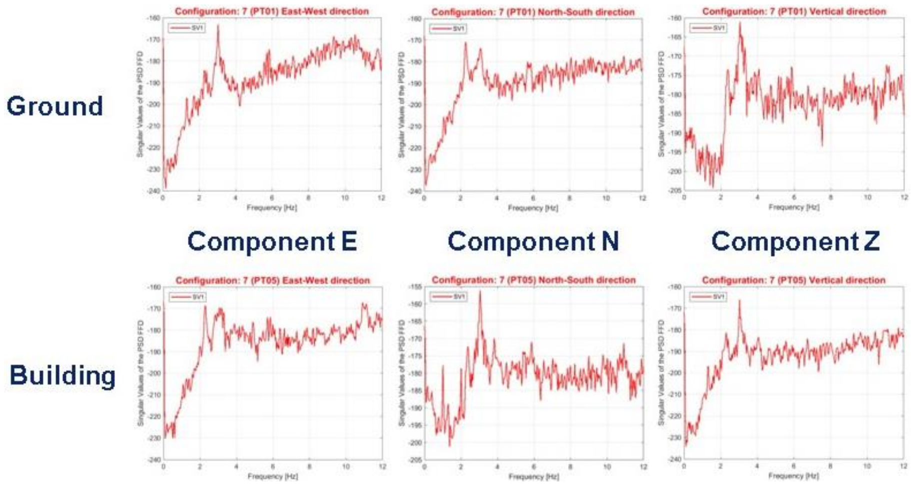

Fig. 7. Ground vs. building accelerometers.

6 Numerical modelling

At the present stage of analysis, the following simplifications in the preparation of the finite element model of the structure have been assumed.

- The actual settlement (rigid rotation) of blocks 1 and 5 was not considered;
- The degree of interlocking between orthogonal walls was neglected;
- The structural seismic joints between blocks were considered as perfectly effective. Therefore, the possible interaction (hammering) between adjacent blocks during the seismic event has been neglected and the blocks have been modelled separately;
- Soil-structure interaction has not been considered.

The basic hypotheses formulated above have made it possible to model a structure that differs from the real one, but have not at all compromised the methodology for assessing seismic vulnerability, which remains unchanged.

In the beginning a 3D frame in AutoCAD environment has been created, representing the fixed wires of the structural elements. This was then imported into the SAP2000 FE code in which each line represents a frame type element. Then, the fixed constraints to the base, the rigid floor diaphragms, defined the materials, the sections and assigned the loads have been assigned [5].

The building has been modelled following the “equivalent frame” approach (Fig. 8). Therefore, by means of the “rigid end offset” command and on the basis of Dolce's theory, the zones with rigid behaviour in the masonry walls and in the floor strips have been defined.

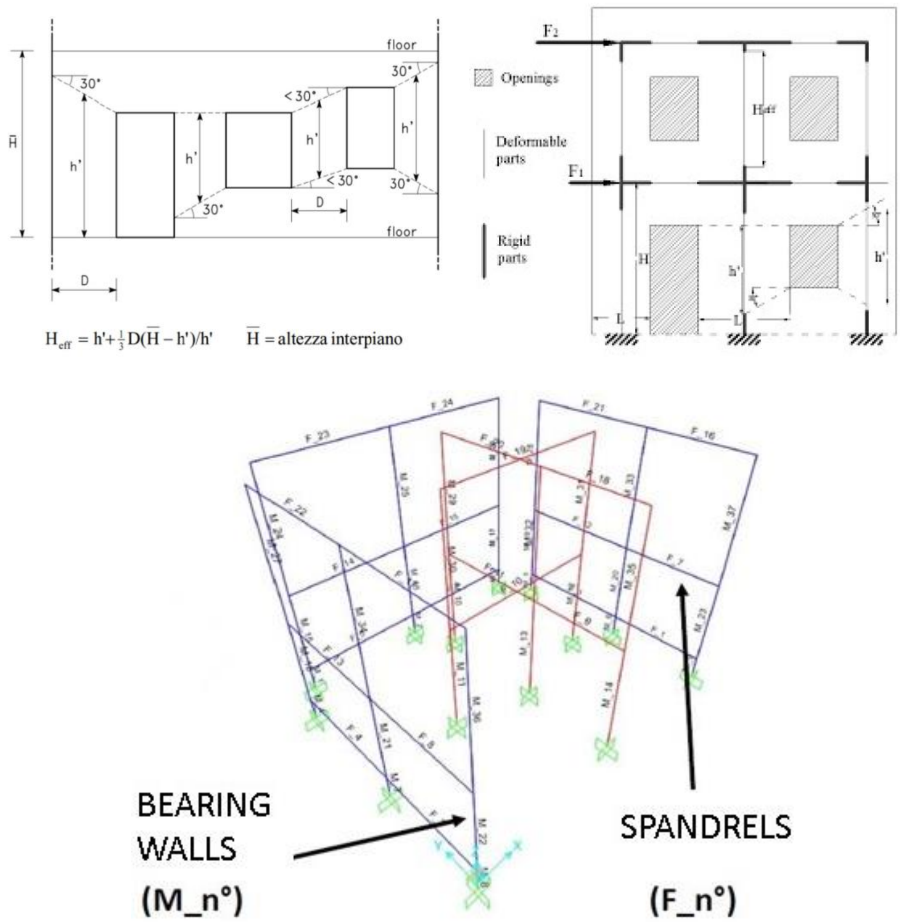

Fig. 8. Equivalent frame model.

SAP2000 does not provide for the automatic creation of plastic hinges for masonry elements. Therefore, to take into account the non-linear response of the structure, ad hoc plastic hinges have been designed, using an elasto-plastic shear-displacement capacity model, in which the ultimate shear is the minimum among those resulting from the 3 typical collapse mechanisms of wall panels: rocking and toe crushing, diagonal shear, and sliding shear failure (Fig. 9).

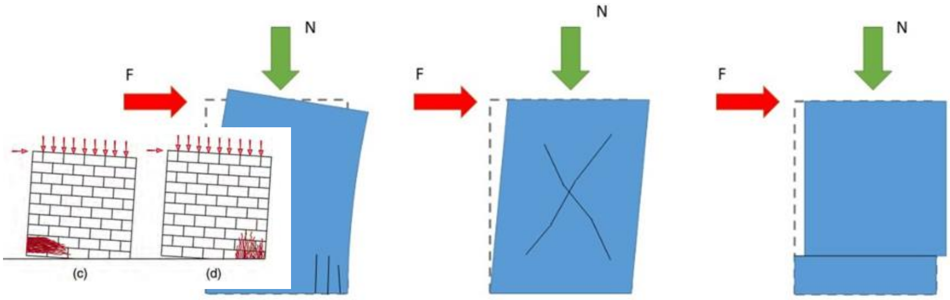

Fig. 9. Failure mechanisms

After model preparation, Modal analysis, Linear dynamic analysis ad Pushover analysis can be considered. Specifically, in addition to the vertical loads, two distributions of inertia forces have been assigned: those called Group 1, proportional to the shear forces at each storey, and those called Group 2, proportional to the floor mass. The two distributions were applied in the 2 main directions, both with positive and negative signs, for a total of 8 pushover analyses for each block.

In Fig. 10, an example of capacity curve of the MDOF system is depicted, expressed as a function of the shear at the base and the displacement of one or more control points, with a representation of the activated plastic hinges.

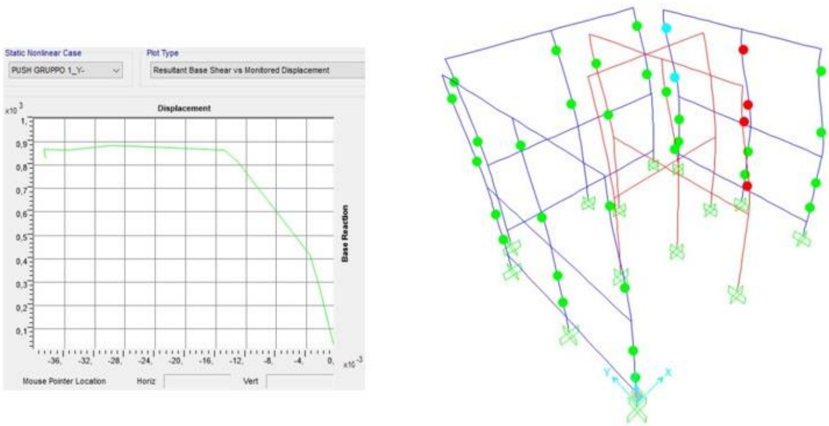

Fig. 10. MDOF capacity curve and plastic hinges evolution.

After that, the capacity curves provided by SAP2000 are reduced to equivalent SDOF systems, scaling them by the modal participation factor Γ referred to the specific load profile. finally, the equivalent bilinear curves have been evaluated using the equal energy rule. Obviously, among all the curves, the one that represents the capacity of the structural system is that one with the lowest maximum shear.

7 Seismic vulnerability assessment

The ζ_E seismic vulnerability index, in agreement with the Italian code, can be expressed as the ratio between capacity and demand in terms of Peak Ground Acceleration [2,3].

The compute indices of all the blocks result higher than unit: i.e., 1.06 for Blocks 1 and 5, and 1.20 for Blocks 2,3,4; with a Demand PGA of 0.1814 g. Thus, the structural model complies with the safety requirements set by the NTC2018 with regard to seismic actions.

8 Conclusions

The Structural Health Monitoring performed on the structure allowed to understand the importance of the soil-structure interaction on buildings with high stiffness and low ductility.

Recording the accelerations at the ground and comparing them with the ones recorded in the building it was possible to understand that the structure behaves as a rigid body.

This work lends itself to being a starting point for further studies, which involve the development of more refined structural models that include the soil-structure interaction (e.g., Winkler soil model).

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