THE ROLE OF THE KNOWLEDGE LEVEL IN THE ASSESSMENT OF SEISMIC PERFORMANCE OF MASONRY BUILDINGS

A thesis submitted to the University of Camerino for the degree of Doctor of Philosophy in

Science and Technology - Computer Science

By

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ABSTRACT

The evaluation of the seismic safety of existing buildings is one of the themes of great importance within the seismic risk prevention and a matter of ongoing research in the modern seismic engineering.

Engineering evaluation of existing structures requires a fundamental knowledge of the materials involved, as-built condition, construction quality, and the extent of any deterioration or distress.

Masonry structures are very heterogeneous and their actual composition is generally unknown.

The current approach adopted by standards or guidelines, to account for the uncertainties involved in the seismic assessment of existing masonry buildings, is in the framework of a deterministic procedure.

It is based on the definition of a discrete number of knowledge levels and on the application of a confidence factor to one specific parameter, usually assumed *a priori* by the code. [9]

The actual approach is rigid, since it does not consider the different specificities of the buildings under examination, and it is conventional, since it does not allow the differentiation of the result of safety evaluation when the performed investigations are diversified.

The most innovative probabilistic approach from the scientific literature is provided by the Guidelines CNR-DT 212/2013.

Through the introduction of a codified use of sensitivity analysis, allows the identification of the parameters that most affect the structural response and aims to limit the inescapable uncertainty.

The research concerns the role of the knowledge level on the assessment of the seismic performance of constructions.

The research focused on historical constructions, where many difficulties arise in the characterization of wall materials, which are generally not homogeneous, and in the efficiency of the connections among structural elements.

As a case study was selected an existing masonry building seriously damaged by the “Emilia

Romagna – 2012" earthquake.

Some original results were obtained from the interaction between traditional tests and dynamic tests.

Traditional tests are mainly oriented to provide local information regarding stiffness and strength of materials and structural elements, while dynamic tests based on ambient vibrations provide information about the global response of the building but they only describe the linear response under low energy inputs.

In particular, the reduction of uncertainties in FEM model modal behavior was achieved by calibrating and upgrading the stiffness of the diaphragms while ambient vibration measurements (Operational modal analysis technique) were used to assess the correspondence of modal parameters of the building.

Moreover, in order to increase the knowledge level necessary to define the parameters that mainly affect the non-linear seismic response of the structure, the results of minor and non-destructive tests were used.

The comparison between the level of safety achieved by the deterministic procedure and that obtained by the probabilistic procedure has yielded very significant results.

Finally, an interesting comparison was made between the seismic response obtained by the FEM model and the damage experienced following the seismic events 2012 in Emilia.

Keywords: Sensitivity analysis, Confidence Factor, Existing Buildings, Seismic Assessment, Dynamic monitoring, Automatic OMA, Vibration-based damage detection, Dynamic identification.

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CONTENTS

1. INTRODUCTION

1.1. Background

Cultural heritage, intended as "every material and immaterial evidence of the cultural identity of a population" is of crucial importance in maintaining and promoting cultural identities and differences of people.

Architectural cultural heritage is surely one of the most important evidence of people cultural identity; along with most famous monuments that are visited every day, millions people still live in ancient buildings and towns.

European Community policies, stated in Europe 2020, recognizes this as a potential instrument of progress and cooperation and as a primary component of the quality of life of citizens.

Among the various priorities, Horizon 2020 definitely devotes special attention to those aimed at promoting actions on cultural heritage.

Cultural heritage is threatened by many enemies, both natural (e.g. ageing, earthquakes, floods) and anthropic (e.g. pollution, vandalisms or absence of maintenance) and its safeguard is a typical issue that can be faced only by a multidisciplinary approach in which both humanistic and technical-scientific expertise are involved.

In this framework, diagnostics occupies a prominent position among actions that can be carried out by scientists and technicians; understanding hazards and vulnerability of a construction, by identifying the underlying causes of the pathological phenomena of which buildings are affected, are basic for risk definition and thus for decision making. [1]

Given the enormous value of the objects under examination, as any restoration or strengthening of a building is invasive at a certain extent, diagnosis of an historical construction needs to be carried out both in the short term, to assess its actual conditions, and in the long term, to assess the modifications in time of the conditions.

These information are crucial in defining the level of intervention and its timing.

In the short term, effective on-site testing programs, which can involve the application of different test methodologies as a combination of destructive tests, minor destructive tests and non-destructive tests, needs to be performed. [2]

In the long term, data of a comprehensive set of physical, environmental and structural variables need to be monitored; very powerful sensors are available for a plethora of physical quantities [3] and using of smart remote sensing [4] enormously reduce the cost for continuous structural monitoring with respect to conventional cabled systems.

Among various sensors, those that allow vibration measures (e.g. accelerometers) are the most used for the health structural monitoring because they allow to perform experimental modal analyses that, repeated over the time, give important information about the degradation of a construction without any invasive intervention. [5]

The philosophy of the knowledge phase, necessary for the best intervention on an architectural cultural heritage, has been also thoroughly addressed in guidelines issued by the Italian Ministry of Cultural Heritage. [7]

One main feature distinguishing the assessment of existing buildings from the design of new ones is that many epistemic uncertainties, due to the limited knowledge and reliability of models, add up to the aleatory ones.

Thus, it becomes crucial to have, on one hand, effective procedures to optimize the investigation protocol in term of cost-invasiveness-benefit and, on the other one, reliable approaches to account for the residual uncertainties in the final assessment.

The common approach adopted by standards or guidelines for the seismic assessment of existing buildings [e.g. at international levels, Eurocode 8 2005 [13] and ASCE/SEI 41/13 2014 [10]] does not explicitly account for the probabilistic issues of the problem, being in the framework of a deterministic procedure (at least concerning the capacity).

For example the Italian standards for construction NTC2008 [6], recently updated by NTC 2018 [29], introduced provisions to define level of knowledge on the basis of destructive and non-destructive tests and the application of a Confidence Factor to one specific parameter, usually assumed a priori by the code. [9]

In the specific it is based on the definition of a discrete number of Knowledge Levels (KL), achievable as a function of information gathered to overcome the incomplete knowledge, and on the application of a Confidence Factor (CF) to one specific parameter, assumed a priori by the code as being the most critical in affecting the outcome of the assessment.

The CF aims to take into account the evaluation of parameters to be adopted in the analysis that could be biased in presence of an incomplete knowledge.

Several critical issues have been raised by various authors on the current approach proposed by codes, concerning both the method for the as-built information step and the meaning of CFs [e.g. in Franchin 2010 [14], Jalayer 2011 [15], Tondelli 2012 [17]].

In particular, numerical simulations of the entire assessment procedure have been carried out both on reinforced concrete [Franchin 2010] [14] and masonry [Tondelli 2012] [17] structures, showing that sometimes the actual code-based procedure may lead to unsafe results.

The alternative would be to frame the problem by including the propagation of uncertainties

(epistemic and aleatory) within a probabilistic approach for the performance-based assessment of existing buildings [as proposed in SAC FEMA – Cornell 2002 [12] and, more recently, in CNR DT212 2013 [8]].

Although such approach is certainly rigorous and represents the actual trend at research level, it requires a higher computational effort and, in addition, it is still not widespread in the engineering practice.

Thus, still within the context of a CF-based approach, CNR DT212 2013 [8] proposes several and significant modifications to the procedure currently adopted in codes in order to overcome some of the drawbacks discussed above [Cattari 2015a] [11].

The most distinctive feature of the new procedure is the introduction of sensitivity analysis as essential tool for a reliable seismic assessment of existing buildings.

In particular, its use is codified and explicitly implemented within the assessment path, that is how to perform it and what to do with obtained results.

The guidelines propose how to implement the sensitivity analysis in a systemic way, in order to:

i) identify the parameters that most affect the structural response allowing to optimize the investigation plan;

ii) explicitly include in the methodological path the evaluation of aleatory and epistemic uncertainties, as well as the model error;

iii) properly select the parameter (or set of correlated parameters) for the application of CF and calibrate its value. [9]

At what extent a non-destructive test campaign can substitute a destructive test campaign?

What is the best compromise between these in order to achieve a reliable knowledge of the assets to give a probabilistic sense to a structural assessment? [2]

Rigorous probabilistic approaches may surely give answers to these questions but also they need to be checked by means of field experiences through the application to significant case studies.

1.2. Outline of thesis

In this thesis the problem of the seismic assessment of existing masonry buildings is addressed, with particular attention to the various sources of uncertainty associated with it.

Generally, many uncertainties affect such knowledge and this may strongly influence the probabilistic assessment of the structural safety.

An accurate seismic verification of the masonry building requires the consideration of all possible different sources of uncertainty and their effect on the seismic answer.

In chapter 2, the procedure reported in the recommendations of the CNR-DT 212/2013 for evaluating the seismic performance of buildings accounting for the “role of the level of knowledge” of the building in a proper and innovative way, are briefly presented and discussed.

Particular attention is given to how uncertainties in engineering safety problems and decision making under uncertainty are dealt with.

Chapter 3 summarizes the investigation tests commonly adopted in the professional field and that can be performed in masonry buildings in order to define the mechanical properties of the masonry.

In chapter 4, the procedure described in chapter 2 was applied to a complex case study seriously damaged by the earthquake "Emilia Romagna – 2012": Palazzo Boldi.

In particular, after a brief introduction to the case study(section 4.1) and after a brief description of the biography of the architect who designed it(section 4.2), the modelling of the work is described step by step with an analysis of the difficulties encountered and how these were overcome(section 4.3).

After conducting the modal analysis on the preliminary model using rigid and deformable slabs in section 4.5, the innovative choice to use Operational modal analysis(OMA) to calibrate the structural behavior of the building proved to be of crucial importance(sections 4.6,4.7).

The procedure reported in the CNR-DT 212/2013 applied to the updated model is based on the sensitivity analysis that guides the choice of in-situ characterization tests to improve the knowledge level of the structure(sections 4.8,4.9).

At the end of the investigations, the group of aleatory uncertainties which most significantly affect the seismic performance of the building was identified and only the confidence factor was applied to it(section 4.10).

In the case of epistemic uncertainty, at the end of the investigations the most reliable of the considered alternatives was chosen.

The final seismic safety evaluation was assessed adopting a final model with updated parameters.

The comparison between the level of safety achieved by the deterministic procedure and that obtained by the probabilistic procedure has yielded very significant results, explained and commented on in an exhaustive and critical way in section 4.10 and in the final conclusions.

In the section 4.11 an interesting section was reserved for comparison between the seismic response obtained by the FEM model and the damage experienced following the seismic events 2012 in Emilia.

Finally, Chapter 5 includes conclusions and suggestions for further work.

2. METHODOLOGICAL PRINCIPLES OF THE SENSITIVITY ANALYSIS IN THE SEISMIC ASSESSMENT OF EXISTING MASONRY BUILDINGS FOR DEFINING PROPER CONFIDENCE FACTOR

2.1. Current format of CF-based procedures proposed in codes

According to well recognized standards at international level in the field of the assessment of existing buildings, like as the Eurocode 8 [2005] [13], at European scale, and the ASCE/SEI 41/13 [2014] [10], at American one, a subdivision in three different Knowledge Levels (KLs) is usually adopted.

Such KLs are differentiated depending on the amount and quality of collected information, which are usually related to:

1) geometry;

2) structural details (indicated as “condition assessment” in ASCE/SEI 41/13) [10];

3) material properties.

In most cases the reaching of a certain KL implies an equivalent state of knowledge on all different abovementioned aspects: for example, in Eurocode 8 [13] the level of completeness associated to three aforementioned levels is classified as limited, extended and comprehensive.

Then the obtainment of a certain KL, through an appropriate investigation plan, leads to the assumption of the corresponding Confidence Factor (CF) value (set in the range of 1.35 to 1.0) and, in some cases, to some limitations on the method of analysis that must be used.

In general, CF must be applied to the parameter selected a priori by each standard and implicitly identified as that mostly affecting the structural response.

Some distinctions are introduced in these documents as a function of the failure mode occurred in masonry panels (if classified as ductile or brittle, that is deformation or force controlled).

In ASCE/SEI 41/13 [10], in the case of deformation controlled mode (prevailing rocking behavior) CF is applied to the drift limit, whereas in the case of force controlled mode (diagonal shear behavior) it is applied to strength parameters.

In the case of Eurocode 8 [13] all the considered failure modes of masonry panels are classified as ductile, by introducing a proper different value in terms of drift limit: despite this, CF is applied only to strength parameters.

It is worth noting that in the case of local mechanisms associated to a prevailing out-of-plane behavior of masonry, the Italian building Code NTC 2008 [6] (recently updated by NTC 2018) [29], that presents a general framework common to that of Eurocode 8 [13], advises to apply CF directly to the structural capacity; this is due to the fact that usually strength parameters do not influence so much the capacity, which is mainly related to geometry and constraints.

A more detailed review of analogies/differences in such codes is presented in [Cattari 2015a] [11]. In general, main drawbacks of the current approach based on the use of CF can be summarized as follows:

- in most cases, a given KL is assigned to the whole building, thus implicitly assuming that sensitivity to all groups of parameters is equivalent; on the contrary, it would be advisable to reduce CF even if some parameters are not investigated in an extended or comprehensive way but if the sensitivity is low;

- the CF is conventionally applied to a predetermined parameter: depending on the properties of the structure, this assumption should be verified by a sensitivity analysis;

- the value of CF is conventionally proposed as a function only of the reached KL: while it should be related both to the variability of the parameter, in the case of an incomplete knowledge, and to the sensitivity of the response to the parameter itself.

2.2. Basic steps of the procedure proposed

With respect to the current procedures based on the use of CF, the most innovative aspect of the procedure herein proposed is the introduction of a codified use of the sensitivity analysis.

This procedure has been originally developed within the context of the PERPETUATE project [Lagomarsino 2015] [16] focused on the protection of masonry monumental buildings, but its principles are general and applicable to any type of existing buildings.

In particular, it allows improving some fundamental issues such as:

- to identify the parameters that most affect the structural response allowing to optimize the investigation plan and strengthen the link between knowledge and assessment;

- to explicitly include in the methodological path the evaluation of aleatory and epistemic uncertainties, as well as the model error;

- to properly select (instead of a priori) the parameter (or set of correlated parameters) for the application of CF and calibrate its value (instead of assuming it conventionally).

The method, instead of assigning a given KL for the whole building, defines which KL (still graduated into three levels) should be achieved for each single parameter, calibrated on the basis of the actual sensitivity of the seismic response to it.

The sensitivity is assessed with respect to a selected Structural Performance Indicator (SPI). Among the different possible choices and according to the final aim of the seismic PBA, the maximum Intensity Measure compatible with the fulfillment of performance levels (IMPLi) has been selected as SPI.

The sensitivity can be computed according to nonlinear static procedures based on over-damped or inelastic spectra.

The IMPLi represents the mean value of this variable and is obtained by adopting for all parameters their mean values: being in the context of a semi-probabilistic procedure, the actual dispersion of parameters is not explicitly considered.

Hence, CF is applied to take into account the uncertainty in the estimation of the mean value of the selected parameter.

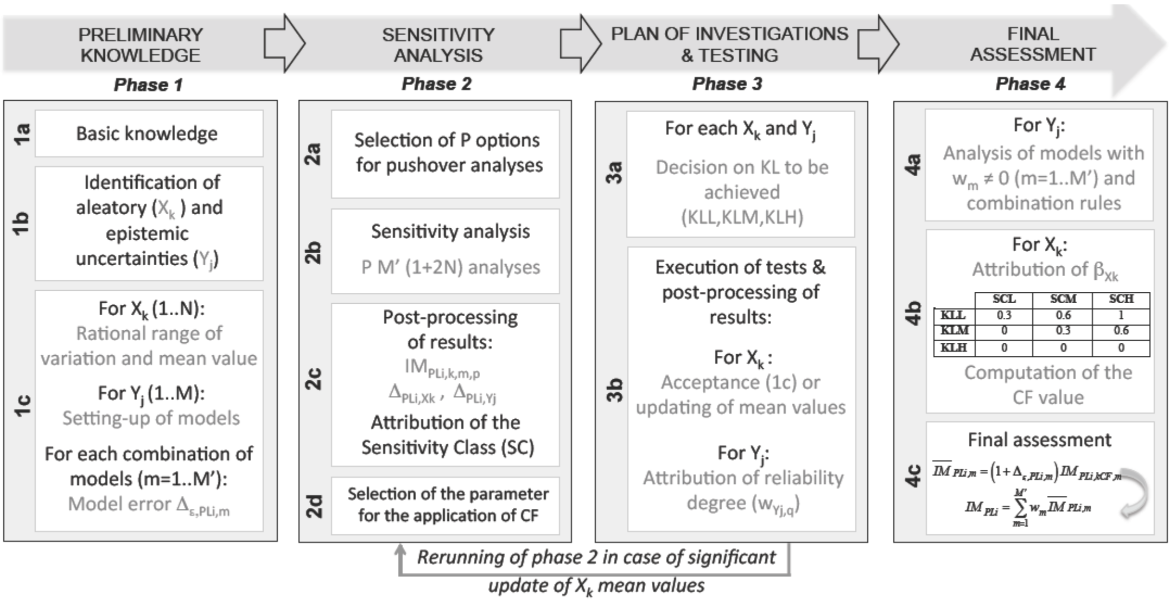


Figure 1. Flowchart of the proposed procedure

The proposed procedure is explained in detail in the following sub-chapters.

2.2.1. Preliminary knowledge

The preliminary knowledge is addressed to the achievement of a basic knowledge level and identification of all uncertainties involved in the building response.

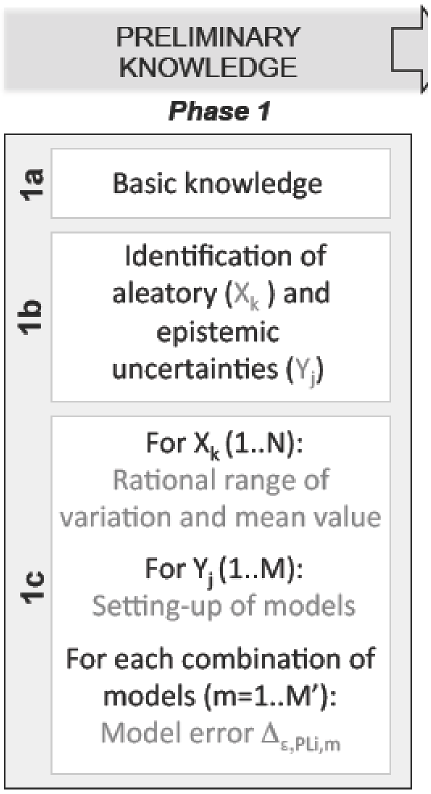


Figure 1-a. Phase 1: preliminary knowledge

It requires the following sub-steps:

1a) - Achievement of a basic knowledge level: it is addressed to preliminarily identify the most suitable model (or models) to be adopted for the seismic assessment and collect all necessary data for the analyses.

1b) - Identification of aleatory and epistemic uncertainties, which are related to parameters involving geometry, mechanical parameters and structural details as well.

Aleatory uncertainties are associated to parameters that are treated as variables Xk (k=1..N, where N is the total number parameters or groups of parameters).

Epistemic uncertainties are usually related to constructive or modelling factors Yj (j=1..M), which are treated through the logic tree approach.

The former parameters might vary in a defined range.

Each factor Yj leads to the adoption of two or more possible models (q=1..mj); the number of possible alternatives may be different for each factor.

If only two alternatives (mj=2, ∀ j=1..M) are considered for each factor (quoted as A and B), 2^M models, obtained by the factorial combination of all possible configurations, have to be considered: each one of them may be identified by a specific sequence of letters given by the corresponding choice on the j-th factor (e.g. in case of M=2: AA, AB, BA and BB).

In the following, the possible combinations (M') are synthetically identified by the counter m (m=1..M', where M'=2^M only if two alternatives are considered for each Yj factor).

1c) - For each variable X_k: identification of a rational range of variation, that is a lower and upper bound ($X_{k,low}$ and $X_{k,up}$) of the mean value of the parameter.

Although the method proposed does not strictly require any probability distribution for random variables, if these are available for a wide population, the definition of the interval can refer to one standard deviation confidence levels.

Once the range of variation is specified, it is possible to define:

$$\bar{x}_k = \frac{x_{k,low} + x_{k,up}}{2}$$

$$f_k = \frac{x_{k,up} - x_{k,low}}{x_{k,up} + x_{k,low}} \qquad \textbf{(1)}$$

where $\bar{X}_k$ is the plausible mean value and f_k will be used to calibrate the CF on the basis of the actual variation expected for each parameter.

For each combination of factors Y_j (M' in total): setting-up of the model.

For the model error $\Delta_{\varepsilon,PLi,m}$:attribution of its rational estimate.

$\Delta_{\varepsilon,PLi,m}$ refers to the evaluation of each PLi by the model corresponding to the m-th combination of the Y_j factors.

It is assumed as a percentage of variation of the expected mean value of the actual seismic capacity in comparison with the value IM_{PLi} given by the model; hence, this parameter should be considered when the adopted model is clearly and systematically on the safe side ($\Delta_{\varepsilon,PLi,m}>0$) or to the detriment of safety ($\Delta_{\varepsilon,PLi,m}<0$). It is worth noting that, at present, model error is generally neglected; anyhow, the proposal is to consider it only when it is expected to be relevant in comparison with the effect of other uncertainties.

2.2.2. Sensitivity analysis

The main aim of sensitivity analysis is to identify the parameters/factors that most affect the structural response among those selected at the end of phase 1.

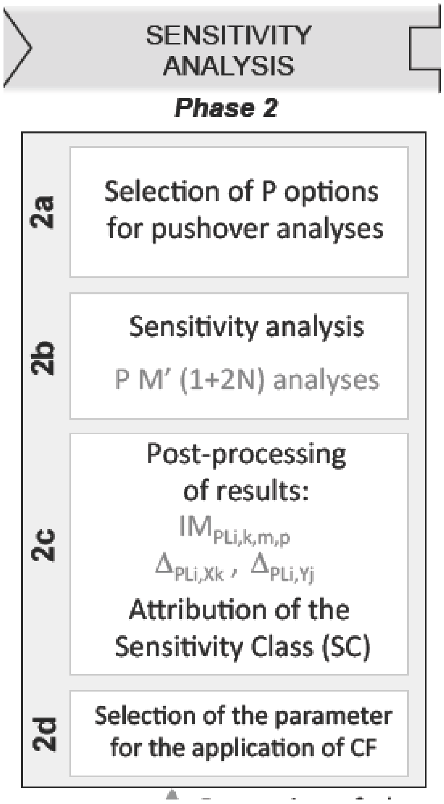


Figure 1-b. Phase 2: sensitivity analysis

2a) - To this aim, the basic tool adopted is the execution of nonlinear static analyses.

In particular, for each m-th model (as a function of the Y_j factors identified), 2N+1 analyses must be performed, that is:

- a first one by adopting as reference for all the parameters the plausible mean value $\bar{X}_k$;

- a set of 2N analyses in which each parameter (or set of parameters) is changed one by one according to the lower ($X_{k,low}$) or higher ($X_{k,up}$) bound of the rational range, as defined in step **3a**.

The execution of a pushover analysis presupposes a choice on many different combinations of conditions related to: the load pattern (e.g. proportional to mass, to the mass and height product or to the first modal shape), the main directions of the building footprint, the positive or negative sense of each direction and the accidental eccentricity (usually proposed by codes as the 5% of the maximum length in the direction orthogonal to that examined).

Although in the final assessment different options have to be considered (as expressly indicated also by standards), it seems worthwhile to select the worst conditions in order to limit the number of analyses to be performed: to this aim, it is useful to perform some preliminary analyses in order to select one or more basic options related to direction, load pattern, accidental eccentricity and control node (enumerated by the counter p=1..P).

2b) - These preliminary analyses may be performed, for each m-th model, by assuming the plausible mean values $\bar{X}_k$ for all variables X_k.

Thus, by considering also the number of models (M') and possible options (P), a total of M'P(2N+1) analyses should be performed.

Indeed, in order to investigate the sensitivity by considering the cross correlation of parameters X_k, it should be more accurate to perform a multivariate second order factor analysis: although certainly more rigorous, it is evident it implies a huge computational effort (2^N analyses rather than only 2N).

If the given PLi is considered, the result of each analysis is $IM_{PLi,k,m,p}$, where the subscript k (related to the k-th parameter) is followed by "-low" or "-up" depending on the assumed value; when for all variables the plausible mean is assumed, this field is replaced by "mean".

Then, for each m-th model and p-th option, it is possible to evaluate the corresponding values of $IM_{PLi,k\text{-}max}$ and $IM_{PLi,k\text{-}min}$ as:

$$IM_{PLi,k_{-\min}} = \min(IM_{PLi,k_{-low}}, IM_{PLi,k_{-up}}, IM_{PLi,k_{-mean}})$$

$$IM_{PLi,k_{-\max}} = \max(IM_{PLi,k_{-low}}, IM_{PLi,k_{-up}}, IM_{PLi,k_{-mean}}) \tag{2}$$

where the subscripts m and p have been omitted in the following for simplicity.

Finally, the sensitivity to variables X_k and Y_j is assessed through the variable $\Delta_{PLi,Xk}$ and $\Delta_{PLi,Yj}$ computed as:

$$\Delta_{\mathrm{PLi,Xk}} = 2\frac{IM_{PLi,k_{-\max}} - IM_{PLi,k_{-\min}}}{IM_{PLi,k_{-\max}} + IM_{PLi,k_{-\min}}}$$

$$\Delta_{\mathrm{PLi,Yj}} = 2\frac{\max(\mu_{j,IMPLi,mean,q}) - \min(\mu_{j,IMPLi,mean,q})}{\max(\mu_{j,IMPLi,mean,q}) + \min(\mu_{j,IMPLi,mean,q})} \quad q=1,..mj \tag{3}$$

Where $\mu_{j,IMPLi,mean,q}$ is the mean of the IM_{PLi} values (computed by assuming the mean value forall random variables) resulting from the branches of the logic tree associated to the q-th option for the factor Y_j.

2c) - Once the sensitivity analyses have been completed and all results post-processed, it is possible to proceed to step **2c**, that is the attribution of a Sensitivity Class (SC), for each k-th parameter and j-th factor (as a function of the i-th performance level).

To this aim, it is necessary to define some conventional criteria for establishing the high, medium and low sensitivity.

A possible criterion for each m-th model is the following:

- firstly,a reference value of the sensitivity parameter $\Delta_{PLi,max}$ is calculated as max[$\Delta_{PLi,Xk}$], by referring only to the sensitivity to variables X_k, taking into consideration the P options for the pushover analysis;

- then, SC to each parameter/factor is conventionally given as a function of $\Delta_{PLi,max}$, for example according to this rule:

<u>High sensitivity</u> (SCH) for $\Delta_{PLi,Xk}$ (or $\Delta_{PLi,Yj}$) > 2/3 $\Delta_{PLi,max}$;

<u>Medium sensitivity</u> (SCM) for 1/3 $\Delta_{PLi,max} \leq \Delta_{PLi,Xk}$ (or $\Delta_{PLi,Yj}$) ≤ 2/3 $\Delta_{PLi,max}$;

<u>Low sensitivity</u> (SCL) for $\Delta_{PLi,Xk}$ (or $\Delta_{PLi,Yj}$) < 1/3 $\Delta_{PLi,max}$.

These ranges could be differently calibrated or established by the seismic assessor.

It is worth noting that the sensitivity class of the k-th aleatory variable could be different for each m-th model, as well as the sensitivity parameter $\Delta_{PLi,max}$ can be very different from model to model.

2d) - The objective of defining sensitivity classes is to identify the need for more investigation for the parameters that most significantly affect the seismic performance of the building.

Thus, in order to overcome some limits noticed on current standards, distinct KLs are planned for each parameter as a function of its specific SC, rather than for the specific "knowledge aspect" as a whole (geometry, material and structural details): this allows to improve the knowledge only where it is relevant.

2.2.3. Plan of investigations and testing

Results from the sensitivity analysis (phase 2) are useful to optimize and reliably plan investigations and tests to be performed (phase 3).

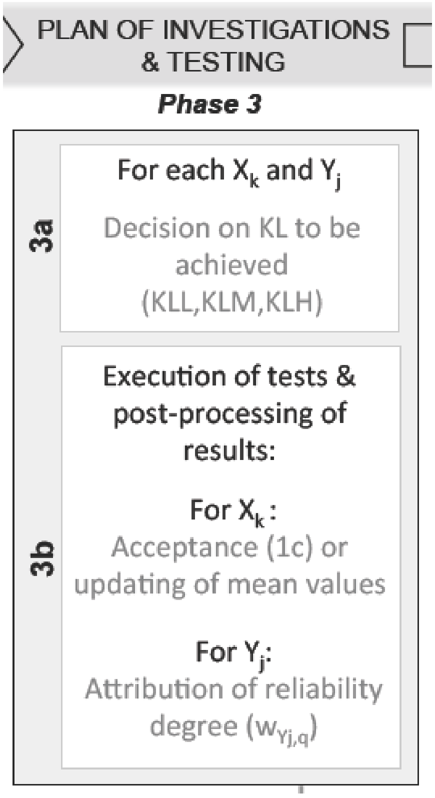


Figure 1-c. Phase 3: plan of investigations and testing

3a) - Regarding the knowledge levels for each single parameter, a division into three levels is proposed, as in Eurocode 8 [2005] [13], which are quoted as KLL (low), KLM (medium) and KLH (high).

Moreover, tools useful to achieve a certain KL are classified as follows:

i) "qualitative" investigations based only on in situ survey, visual inspections, data available from archive records;

ii) "indirect" investigations based on not destructive tests on both materials and structural details (such as pulse sonic tests, thermography etc.);

iii) "direct" investigations based on minor or destructive tests on both materials and structural details (such as coring of samples, double flat jack test, diagonal compression test, endoscopy, etc.).

3b) - The objectives of the detailed investigations are:

i) in case of X_k parameters, to confirm/update the plausible mean value to be adopted in the final assessment;

ii) in case of Y_j factors, to acquire enough data to choose the most suitable model or, at least, to attribute to each one a subjective probability $W_{yj,q}$ ($\sum_{q=1}^{mj} Wyj,q = 1$), related to the level of reliability of each choice.

Then the residual uncertainties are treated:

i) through the application of the CF, in case of aleatory variables (X_k);

ii) through the logic tree approach, in case of epistemic uncertainties (Y_j factors).

Moreover, the model error can also be considered.

In case of Y_j factors, when the final assessment is slightly affected by epistemic uncertainties, it is suggested to make a choice among the alternatives considered (that most conservative – in case of KLL – or that most reliable – in case of a higher KL achieved) in order to limit the final computational effort.

On the contrary, when the SC is higher (SCM and SLH) and the data acquired are sufficient to assign $W_{yj,q}$, the combination through the logic tree approach is advisable to improve the reliability of the PBA.

2.2.4. Final assessment

Finally the final assessment is assessed through the following steps:

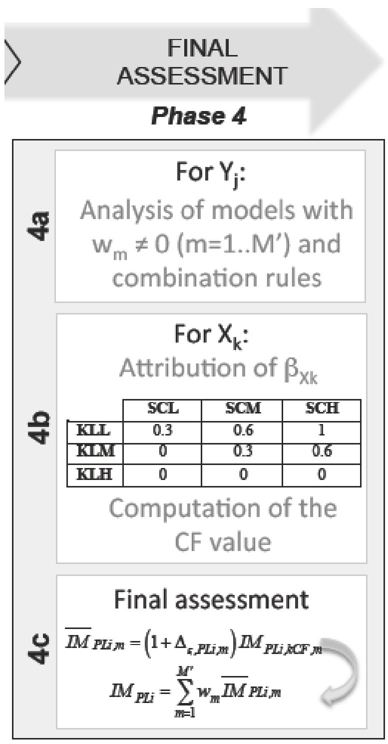


Figure 1-d. Phase 4: final assessment

4a) - Considering epistemic uncertainties, the final selection of models to be adopted and rules for their combinations.

4b) - Considering aleatory uncertainties, the evaluation of the residual incomplete knowledge and computation of CF value to be adopted for each model.

Regarding X_k variables, CF has to be applied to one "main parameter" (or group of parameters) X_{kCF} selected among those associated to the sensitivity class high (SCH) for the m-th model.

The value of the intensity measure of the seismic input ($IM_{PLi,kCF,m}$) that produces the performance level PLi is obtained from the model in which all parameters have been set to the plausible mean value and the CF is applied to X_{kCF}.

4c) - The final evaluation provided by each m-th model (and a given p option for the execution of the pushover analysis) is computed as:

$$\overline{IM}_{PLi,m} = (1 + \Delta_{\varepsilon,PLi,m}) IM_{PLi,kCF,m} \quad \textbf{(4)}$$

where $\Delta_{\varepsilon,PLi,m}$ is the model error related to the m-th branch of the logic tree; it is mainly related to the capacity of model adopted of describing the specific examined asset and should usually assume a negative value or more rarely a positive one.

The evaluation of CF has to take into account:

i) the actual variability of the parameter to which CF is applied, by considering f_k (eqn. **(1)**);

ii) the residual uncertainties associated to the incomplete knowledge process, which is measured by a factor β_m, defined on the basis of different KLs on all parameters.

Hence a β_{Xk} factor is introduced to measure the residual uncertainty on each parameter and ranges from 1 to 0 [Cattari 2015a] [11].

The introduction of such factor aims to guarantee equal percentiles of safe outcomes, independently of the reached KL.

Considering each m-th model, it is possible to assign to each parameter X_k the corresponding $\beta_{Xk,m}$ value.

Hence, the maximum value β_m (β_m=max[$\beta_{Xk,m}$, k=1..N]) is assumed as reference to compute the CF value to be applied to the X_k parameter (or set of parameters) in the m-th model ($CF_{XkCF,m}$) as follows:

$$CF_{XkCF,m} =$$

$$(1+\beta_{\mathrm{m}} f_{kCF,m}) \quad if \quad IM_{PLi,k-\min} = IM_{PLi,k-up}$$

$$(1) \quad if \quad IM_{PLi,k-\min} = IM_{PLi,k-mean}$$

$$(1-\beta_{\mathrm{m}} f_{kCF,m}) \quad if \quad IM_{PLi,k-\min} = IM_{PLi,k-low}$$

(5)

The nonlinear analyses are then performed adopting for parameter X_{kCF} the product of the corresponding plausible mean value $\bar{X}_{kCF}$ by $CF_{XkCF,m}$.

The CF value is defined in such a way to limit the selected parameter within the originally assumed plausible range (the low or up value is used in case of a high SC with a low KL).

Finally, by considering the effect of CF application and the combination of results through the logic tree, for each given p-th options examined, the value of IM_{PLi} is computed as:

$$IM_{PLi} = \sum_{m=1}^{M^I} w_m \, \overline{IM}_{PLi,m} \qquad \textbf{(6)}$$

where W_m represents the weigth of each branch of the logic tree as resulting from the product of weigths associated to the options of Y_j factors that define the m-th model.

3. CHARACTERIZATION OF MASONRY BUILDINGS

3.1. Introduction

The guidelines for assessing and reducing the seismic risks associated with historical structures provide general principles and specific suggestions depending on the structural typology.

As a general rule, interventions must be as limited as possible, and they must be based on increasing levels of knowledge.

The methodological path is summarized in Figure 1.

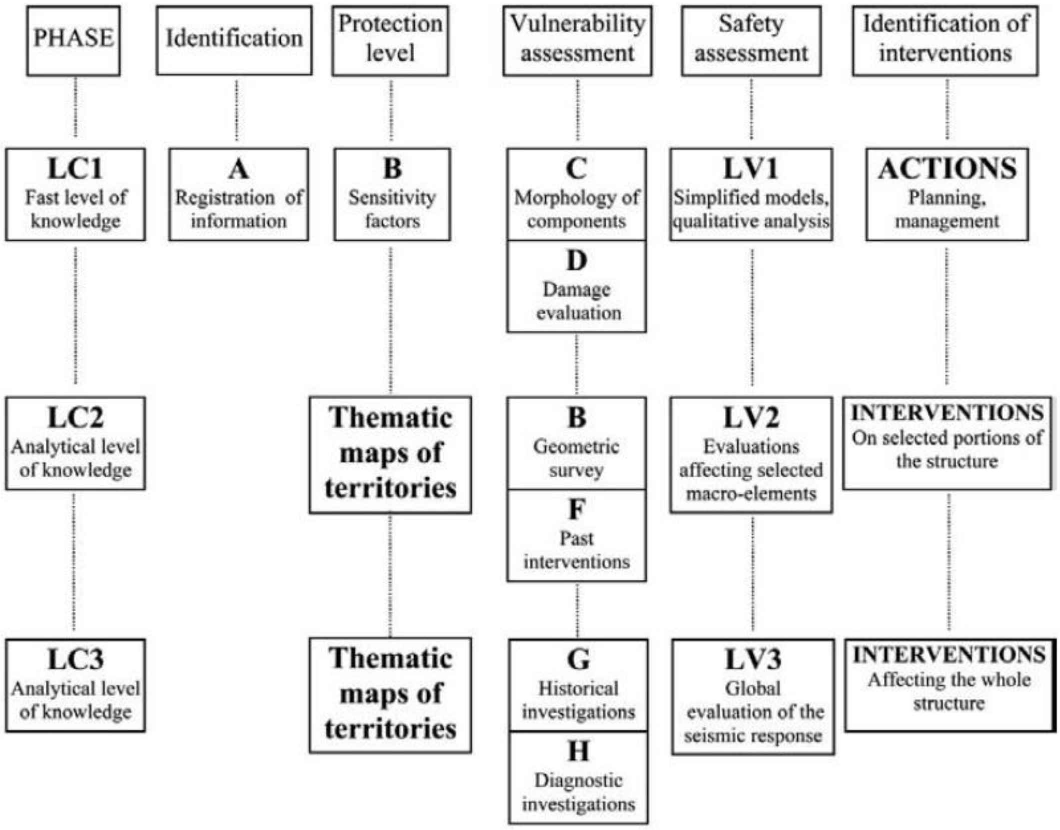


Figure 2. The methodological path for interventions on historical structures

Limited or extensive interventions are possible, but a high level of confidence in the knowledge of a structure's behaviour is required.

As a consequence, a number of tests and surveys must be conducted to define a representative model of the behaviour of a structure or to demonstrate that a global approach cannot be pursued. In the latter case, simplified assumptions on limited parts of the structural system can be used to support decisions related to the extension and nature of interventions.

However, destructive tests must be limited in number due to the valuable characteristics of historical structures.

Conversely, non-destructive and non-invasive tests are preferred. [31]

Engineering evaluation of existing structures requires a fundamental knowledge of the materials involved, as-built condition, construction quality, and the extent of any deterioration or distress.

For many years the traditional approach to obtaining such information was through destructive probes and removal of materials for laboratory testing.

The development of non-destructive evaluation (NDE) methods have changed the way engineers approach structural condition assessment projects.

A wide range of non-destructive and in situ diagnostics are not only available to the practicing preservation engineer, but are becoming a common component of structural evaluation projects.

Preservation engineers must have a basic understanding of diagnostic techniques and the ability to not only recommend complementary test methods but also interpret basic test results.

Engineers that are able to apply NDE methods to preservation projects analyze and design with confidence, thereby ensuring life safety and serviceability objectives are met while at the same time minimizing the necessary level of intervention. [18]

In this chapter have been summarized the investigation tests commonly adopted in the professional field and that can be performed in masonry buildings in order to define the mechanical properties of the masonry.

Before starting the case study investigation, it was become necessary to acquire theoretical notions on the damage diagnosis of masonry buildings as to calibrate the initial parameters without invasive tests.

3.2. Non-destructive tests and minor destructive test

Non-destructive (NDT) and minor destructive testing methods (MDT) are tools of investigation, which can be applied without any or with only small interventions in the object to be examined.

These techniques can give hints to irregularities within the historic masonry structure, which is often inhomogeneous.

Irregularities may derive from differences in material or microstructure, from voids or delaminations, cracks, salt or moisture influence or differences of loading.

Starting at the surface of the object NDT and MDT offer possibilities to border problem areas, to detect structural differences and to amend the reliability of statistic evidence relative to or in addition to selective material extractions and investigations.

Depending on the particular question and methodology NDT and MDT techniques are useful to get a first survey of large areas at the beginning of building or restoration projects namely on structures with defects or damages.

It is then possible to investigate surfaces and parts of protected historic constructions or areas, which are difficult to access, with higher precision.

These techniques can also be applied for long-running observations (monitoring) or be used as quality-assurance after repair interventions and during historical building researches.

Generally NDT and MDT applications are a part of the global investigation of the building.

They do not replace other investigation techniques completely but in the case of historic monuments NDT should be preferred to traditional tests on extracted samples when both types of techniques can solve the problem. [20]

MONITORING:

Sensors to measure movement (tilt, displacement, strain) and environmental conditions (temperature, humidity, wind speed and direction) are often used to track long-term building response. [21]

INFRARED TERMOGRAPHY:

A thermal pulse is applied to a surface causing a non-stationary heat flow.

The propagation of the heat into the body depends on material properties like thermal conductivity, heat capacity and density of the inspected specimen.

If there are inhomogeneities in the near surface region of the structural element this will result in measurable temperature differences in the local area of the surface.

Impulse thermography (IT) and pulse-phase thermography (PPT) are active approaches for a quantitative thermal scanning of the surface of various structures and elements.

The surface of the structure to be investigated is heated by using a radiation source.

After switching off the heating source, the cooling down behavior is recorded in real time with an infrared camera.

While observing the temporal changes of the surface temperature distribution with the infrared camera, near surface inhomogeneities will be detected if they give rise to measurable temperature differences on the surface.

The main approach of impulse thermography in analysing the thermal data is to interpret the function of surface temperature versus cooling time for selected areas with and without inhomogeneities. [20]

For solving the inverse problem, i. e. to get information about the thermal and geometrical properties of the detected defect from the difference curves, numerical simulations can be performed.

Pulse-phase thermography is based on the application of the Fast Fourier Transformation (FFT) to all transient curves of each pixel.

Thus, one obtains amplitude and phase images for all frequencies.

Amplitude images show the internal structure of a specimen up to a maximum available depth depending on the frequency (low pass filter behavior).

Phase images show the internal structure within a certain depth range depending on the frequency (band pass filter behavior).

Active methods have proven their usefulness for locating defects in the near surface region like voids and honeycombing in concrete and delaminations of tiles, plaster and glued carbon fibre reinforced laminates.

Further developments and applications in civil engineering are using the sun as a natural heat source, e.g. for the inspections of bridge decks and of paving in general. [20]

OPERATIONAL MODAL ANALYSIS AND EXPERIMENTAL MODAL ANALYSIS:

Historical structures are characterized by a high level of uncertainty, which affects material properties and structural schemes and is related to deterioration processes or previous interventions and structural modifications.

The level of knowledge can be increased by experimentally evaluating a structure's dynamic properties, and the resultant data can be used to refine and update numerical models that are representative of the real structural behavior.

Moreover, the periodic monitoring of relevant parameters can help identify eventual deterioration phenomena.

Thus, dynamic tests, in conjunction with model updating, are becoming reliable tools for non-destructively assessing historical structures. [31]

Experimental modal analysis (EMA) is a well-adopted method in order to identify the dynamic parameters of the structures and present a mathematical or modal model.

The structural modal model generally consists of frequencies, damping ratios, mode shapes and modal participation factors. Adopting a parametric model from the measured data is known as system identification.

The experimental identification of modal parameters can be traced back to the middle of the Twelve century.

EMA method estimates the modal parameters of structures based on the known artificial input force and recorded output responses.

The input force is applied to the structures by shakers or impact hammers and the output responses are generally measured by accelerometers sensors.

Consequently, EMA is performed in laboratory condition and the experimental instruments and data signal processing algorithms play a pivotal role in modal parameter estimation.

There are some shortcomings within EMA processes especially for civil engineering structures. Most civil engineering structures such as bridges, buildings, etc are under ambient loads like wind, traffic, pedestrian and since these loads are immeasurable, the input loads is not defined exactly. On the other hand, vibrating huge structures by shaker or impact hammer is very expensive and difficult, if not impossible.

These reasons motivated researchers to identify the structures characteristics by considering just the response of the structure, regardless of input loads.

The algorithms estimating the dynamic parameters of structures just based on the output responses became popular as operational modal analysis (OMA) or output-only modal analysis or ambient vibration analysis or in-operation modal analysis.

Primary studies about OMA were established in 1990s.

Researchers, particularly civil engineering community, deeply focused on OMA techniques since about 15 years ago.

Over the years, OMA has evolved as an autonomous discipline and have been attracting great research interest for many years.

It is worth noting that the first book specially deals with OMA, was released in 2014.

The basic equations of OMA algorithm are mathematically similar to EMA methods and most of OMA techniques are the extension of EMA algorithms.

The main difference is that in OMA methods the nature of input force is assumed to be stochastic (white noise), smooth and broadband and it is considered to be uniformly distributed.

It is to be noted that the modal parameter identification of structures is not the main purpose of OMA.

There are several significant applications such as Structural health monitoring (SHM), model updating, sensitivity analysis, force identification. [32]

ULTRASONICS (ECHO AND THROUGH TRANSMISSION):

The method is based on the transmission and/or reflection of ultrasonic waves generated by an ultrasonic transducer or transducer array.

Longitudinal as well as transversal waves can be generated.

The velocity of propagation depends on mechanical parameters of the structure, the reflection on the contrast of the acoustic impedances at the interface.

The principle of the ultrasonic echo technique with separated transmitter and receiver is based on the emission and reflection of impulses generated by a transducer.

Inner voids in a specimen can be regarded as an interface between two different materials (brick/air) for the propagation of sound and lead to total reflectance of the ultrasound waves.

The propagation time of the reflection echo is proportional to the depth of the reflector (assuming a constant velocity of propagation).

For several test problems it is advantageous to use transducer arrays and/or to combine it with a 3D reconstruction calculation (3D-SAFT, Synthetic Aperture Focusing Technique).

For tomographic application the transit time has to be measured in different directions relative to the surface.

The inner structure of the building element will influence this transit time.

In order to measure the transit time most accurately, the first arriving point has to be detected. [20]

GROUND PENETRATING RADAR (GPR):

A relatively modern non-destructive-testing method that can help provide information about subsurface construction is ground penetrating radar (GPR),also known as surface penetrating radar.

This technique transmits pulses of microwave energy (electromagnetic waves) into a material and then monitors for reflections of these waves.

Wherever the wave encounters a significant change in dielectric constant, typically caused by an embedded item or a void, a reflection is visible to the operator.

The depth of the feature can be estimated based on the pulse travel time.

This method is particularly adept for locating air voids and embedded metallic items.

Before-and-after scanning can be used to determine if voids were successfully filled during compatible injected fill (CIF) repairs.

GPR can also be used to locate blind headers, discrete veneer headers, and significant changes in moisture content.

GPR uses very low energy pulses, and it is safe to use in occupied buildings.

The operator can get information along the line of a GPR scan instantly and adjust further investigation accordingly.

The frequencies used for masonry evaluation generally provide useful information to a depth of approximately 24 inches or less.

Because GPR-device output requires significant interpretation by the operator, its effectiveness depends largely on the experience and expertise of the operator. [18]

IMPACT ECHO:

In impact-echo a mechanical point impact is used to generate an acoustical impulse, which propagates into the concrete.

Multiple reflections of low frequency waves between the external resonance frequencies and to evaluate structural integrity.

Impact-echo is a wave propagation-based technique which uses frequency domain analysis for data interpretation.

Frequency spectrum analysis is performed on the waveform obtained from a mechanical impact applied on the surface of the concrete element.

By applying a point impact on the surface of the test object, a transient stress pulse is generated and propagates into the concrete as compressional, shear and surface waves.

The compressional and shear waves, which travel through the material, are partly reflected by any internal interface or discontinuity such as reinforcements, ducts, defects, delaminations.

These waves are almost totally reflected if the second material is air, such as in the presence of a void or at the external boundaries of the element under investigation.

Therefore, the principle of Impact-echo testing is based on multiple reflections of an acoustical wave impulse between the surface and any internal reflector. [20]

VISUAL/OPTICAL TESTING:

An important aspect of evaluating historic structures is understanding the cause or causes of observed distress.

Common types of masonry distress include cracks, spalls, efflorescence, and surface erosion.

Numerous questions can arise regarding the various types of distress and about the best method to use in evaluating distress based on the situation.

One of the most important tools in evaluating distress is visual observation by an experienced investigator.

The cause of many crack patterns or surface erosion patterns can be reasonably deduced based on surface observations alone by such an expert.

Sometimes additional subsurface investigation using non-destructive evaluation, probe openings, or borescope observations may be required to determine subsurface conditions.

Visual observation may also include the installation of crack monitors or tiltmeters to track movement of cracks or walls. [18]

<u>PETROGRAPHY</u>:

If it is possible to obtain a small material sample, the use of laboratory-based petrography to examine the sample microscopically and chemically can provide valuable information about distress mechanisms. [18]

Petrographic examination of mortar and masonry-unit materials may also be useful in determining a general category of performance.

However, particular caution should be used with pressure wave, surface hardness, and petrographic methods to predict masonry strength.

Research has shown poor relationships between these methods and masonry strength properties. Nevertheless, these methods may be able to provide estimates of approximate strength as being either weak, of average strength, or strong.

Often, such simple approximations are appropriate for simple structures or preliminary evaluations. [22]

<u>BORESCOPE</u>:

While not entirely non-destructive, borescopes provide visual verification of internal anomalies detected using non-destructive methods. [23]

This technique allows the viewing of the interior of inaccessible areas by inserting a fiberscope (a bundle of flexible optical fibers) or a borescope (a bundle of rigid optical fibers) into the void.

Both carry the high intensity light along their length.

Some manufacturers have coordinating lines between their "structure scopes" and optional camera equipment.

While the technique is marginally useful for solid masonry structures with limited voids, it has been used successfully on masonry cavity wall construction. [24]

SINGLE FLAT JACK:

The aim of the test is determinate of the state of stress acting in a masonry structure.

The flat jack is a steel pad which is to be inflated with oil until the slot is tied positively, i.e. the original situation is restored, the relative strength can be reconstructed.

The determination of the state of stress is based on the stress relaxation caused by a cut perpendicular to the wall surface; the stress release is caused by a partial closing of the cut slot, i.e. the distance between the edges of the slot after the cutting is lower than before.

A thin flat-jack is placed inside the slot and the pressure is gradually increased to restore the distance measured before the cut. [20]

DOUBLE FLAT JACKS:

The aim of the test is determinate of the deformability characteristics of a masonry and study of the stress-strain behaviour of the masonry.

Two parallel cuts are made in the masonry, at a distance of about 40 to 50 cm from each other.

The two jacks delimit a masonry sample of appreciable size to which a uni-axial compression stress can be applied.

Measurement bases for removable strain-gauge or LVDTs on the sample face provide information on vertical and lateral displacements.

In this way a compression test is carried out on an undisturbed sample of large area.

Several loading-unloading cycles may be performed at increasing stress levels in order to determine the deformability modulus of the masonry during loading and unloading phases. [20]

3.3. Destructive tests

The mechanical characterization of masonry is a difficult task due to the heterogeneous and composite character of the material.

The properties of units, mortar and the composite can be normally obtained in the laboratory by standard destructive experiments that have been devised for new structures.

The experiments proposed by the current standards are hardly praticable in the case of existing buildings, due to the difficulties in extracting representative samples equivalent to the specimens required by regulations, e.g. in terms of dimensions, arrangement of components and integrity.

The case of historical masonry is even more complex, since the structure cannot be excessively damaged during the in-situ sampling due to its cultural, historical and economical value.

The characterization of the mechanical behavior of existing masonry is also possible thanks to the use of methods that are based on the in-situ core drilling of existing masonry members, made of clay brick and low-strength lime mortar.

The proposed destructive tests are suitable for existing masonry structures and especially for those of the built cultural heritage, since a direct estimation of the mechanical parameters can be obtained without damaging excessively the historical structure. [25]

SAMPLING CORES:

An interesting possibility is the extraction of samples to be subjected to destructive testing in the laboratory.

The sampling procedure must inflict the lowest possible damage to the historical structure.

The technique adopted is very important, since the specimens must be as undamaged as possible to be representative of the in-situ material.

The extraction of mortar samples from the joints of a wall is difficult, since the material is brittle and usually crumbles as soon as it is removed from the original location.

The sampling of wall portions, to evaluate the properties of the composite material, is almost impossible for existing historical structures.

Recent studies have shown the possibility of evaluating the mechanical behaviour of existing masonry by core drilling and subsequent mechanical testing of samples in the laboratory.

Drilling is usually horizontal and perpendicular to the face of a structural member, like a wall. [25]

TESTS OF MORTAR SPECIMENS:

The aim of these tests is to obtain the flexural and compressive strength of prismatic specimens, and compressive strength and modulus of elasticity for cylindrical specimens. [27]

Masonry mortars are composed of cementitious materials, aggregates, water, and admixtures when specified.

Cementitious materials include portland cement, masonry cement, mortar cement, slag cement, blended hydraulic cement, hydraulic cement, quicklime, hydrated lime and lime putty.

Aggregates consist of natural sand or manufactured sand.

Admixtures may include such materials as coloring pigments, water repellent agents, accelerators, retarders and air-entraining agents.

Quality assurance testing of site-prepared mortar is fairly uncommon, except on large jobs or for essential facilities.

When mortar testing is required, it is essential that all parties involved possess a thorough knowledge of the mortar specifications, test methods and standard industry practices. Misinterpretations of these standards can result in improper testing and confusion regarding compliance with specifications. [26]

COMPRESSION TEST:

The major goal of the compression test is to determine the compressive strength and the modulus of elasticity of masonry.

The wall specimens are loaded uniformly in compression and the maximum achieved load is recorded.

The characteristic compressive strength of the masonry is derived from the strength of the individual specimens.

If the masonry units, or the mortar, are not capable of achieving the exact specified strength, then it is permitted to adjust the measured values.

During the testing procedure, the testing machines are used to apply load to a specimen, such that displacements are uniformly distributed across the loaded surfaces.

Specimen should be put centrally in the testing machine.

The top and bottom of the specimen have to be in full contact with the testing machine.

Load should be applied uniformly to the top and bottom of the specimen and increased constantly.

The compression force is applied in three equal stages up to 50% of maximum estimated force, in order to determine the modulus of elasticity.

After each step, the compressive force should be kept constant for 2±1 min in order to determine the changes in height.

After the completion of the measurement in the last step, the compressive force should be increased at a constant rate until failure of the tested specimen.

In order to measure the modulus of elasticity, displacement measurements should be taken at the four measuring points up to about 50% of the maximum load. [27]

DIAGONAL COMPRESSION TEST:

The diagonal compression test is carried out with a procedure which provides the accurate means to measure the diagonal tensile (shear) strength of masonry walls.

In fact, the masonry assemblages will be loaded in compression along one diagonal of the specimen, causing a diagonal tension failure with the specimen splitting apart parallel to the direction of the load.

Actually, the specimens will be placed into the testing machine with diagonal axis position.

The load on the specimen will be increased until failure of the specimen occurs.

Treatment of the load should be in suitable increment rates. [27]

4. CASE STUDY: “PALAZZO BOLDI” – VIADANA (MANTUA)

4.1. Introduction

In the following section, the application of the proposed procedure to a real building located in Viadana (Mantua), and seriously damaged by the earthquake of 20th May, 2012 (ML=5.8, depth 10 km with epicenter near Finale Emilia) and of 29th May, 2012 (ML=5.6, depth 8 km with epicenter near Medolla) is presented.

This historic building is one of the most important buildings in the city of Viadana.

It consists of three floors of brick masonry and lime mortar and its construction dates back to about 1750.

The horizontal elements are alternately timber floors or steel beams and hollow flat block with or without concrete(most at the upper floors) and brick vaults (generally at the lower floors), while the roof is composed of timber elements.

The staircases are made of bricks or concrete.

The building is characterized by a “U” shape widely used for the stately homes of these years.

The unusual raising of the ground floor from the ground level probably served for the function of inserting the access solution with a double flight of stairs.

The internal distribution of the rooms respects the “en enfilade“ type, at least for the rooms on the main front.

The access to the inner courtyard is enriched by the portico, whose axis of symmetry is underlined.

The central portico is composed of two columns surmounted by a balcony that is identical to that of the entrance.

Figure 3-a. Geographical location of the building

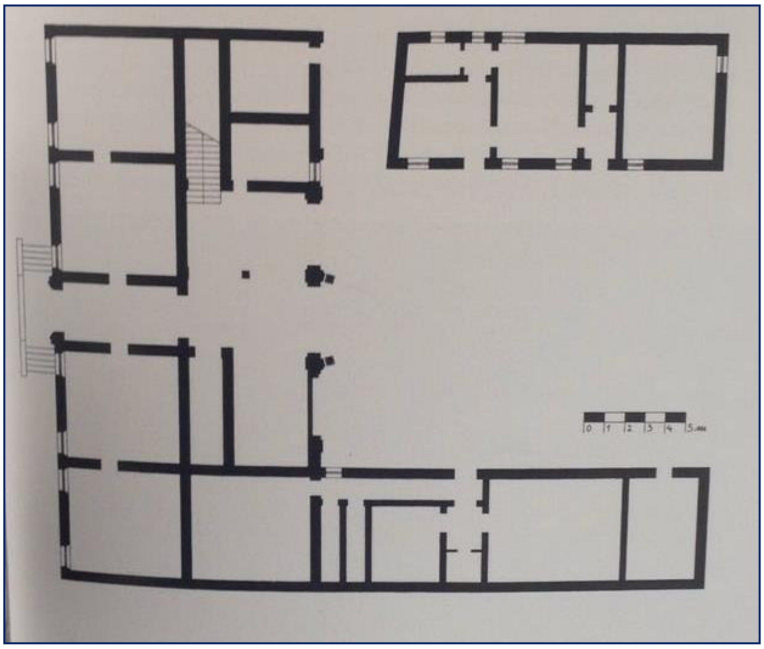


Figure 3-b. Palazzo Boldi – ground floor plan

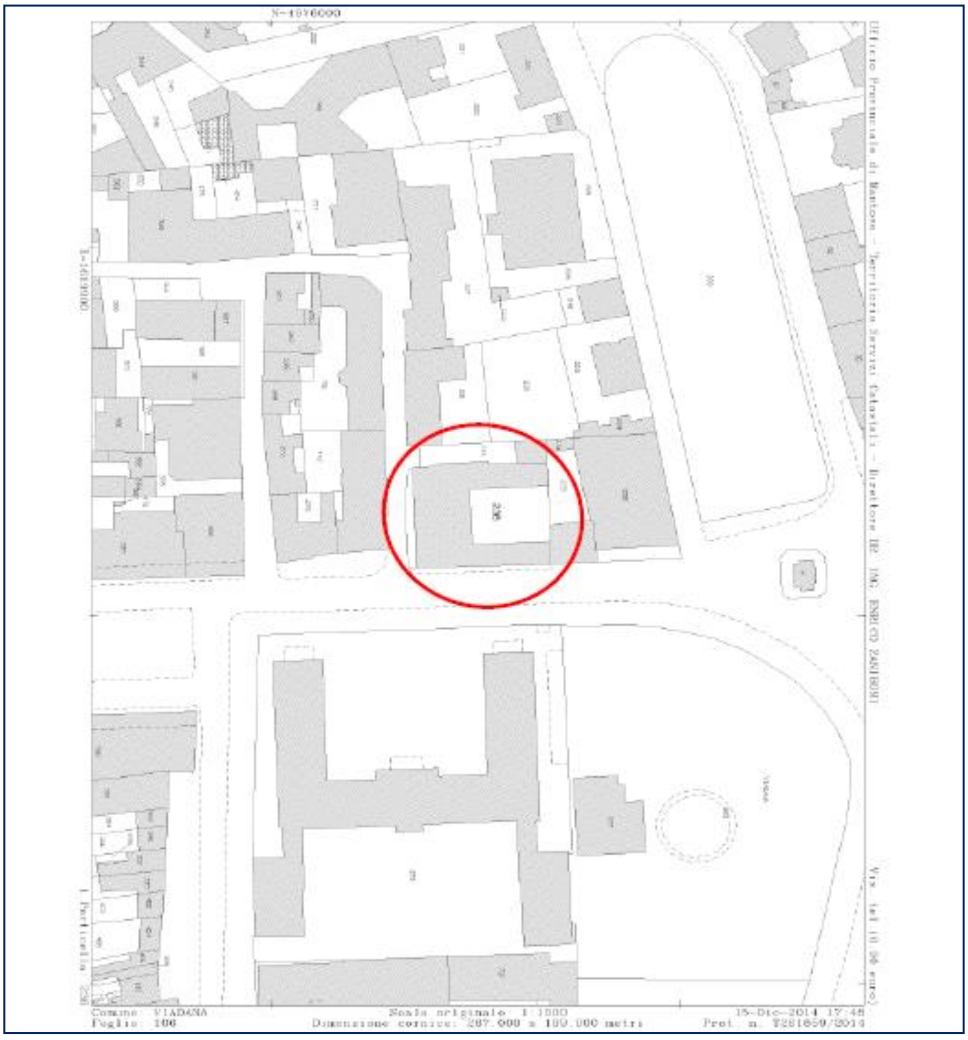


Figure 4. Cadastral location of the building

Figure 5-a. Palazzo Boldi – south elevation

Figure 5-b. Palazzo Boldi – west elevation

Figure 5-c. Palazzo Boldi – east elevation

Figure 5-d. Palazzo Boldi – north elevation

4.2. Pietro Antonio Maggi: architect biography

Pietro Antonio Maggi was born in Viadana(MN) in 1709.

He was one of the greatest representatives of the Emilian-Lombard late baroque.

Before becoming an architect he worked as a plasterer and decorator.

Subsequently he issued specific technical reports and managed sales and rental of real estate.

He was a “practical” architect that was formed in the construction site.

His works are characterized by geometric simplification and the functionality of space.

Method, rigor, sociality, serenity and kindness are the strengths of the architect that we also find in the buildings designed by it.

He made his skills available to serve the community of Viadana, indeed his work is concentrated in Viadana and the surrounding countryside.

Unlike other urban centers around Mantua, Viadana did not develop around the affairs of large landowners or religious institutions.

Its position, at the center of a capillary system of waterways, had made Mantua a place of exchange and commerce and a center of almost forced passage for those who were headed towards Milan or Venice.

In Viadana, a rich middle class, engaged in the judiciary and public offices, in commercial and entrepreneurial activity, seems to be in the process of consolidation.

To create suitable buildings for representatives of the high society of Viadana, there was the necessity for a professional in a position to mediate the two tendencies and to supply technically valid solutions.

The figure of Pietro Antonio Maggi perfectly responded to these characteristics in which technical skill and compositional correctness were perfectly combined.

This is clearly visible in all his works, from the public buildings to the private residences like Palazzo Boldi.

Pietro Antonio Maggi died in Viadana(MN) in 1770.

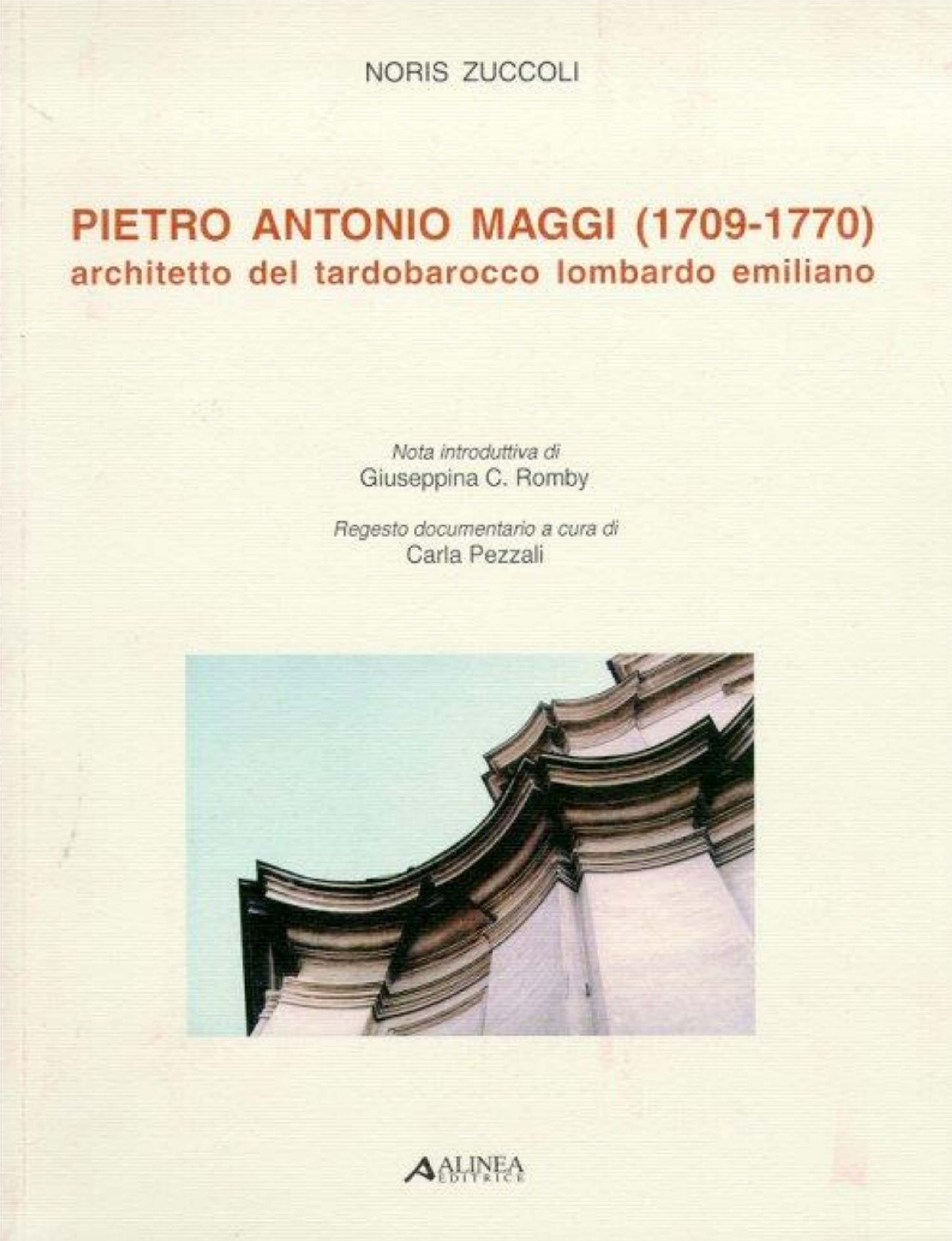


Figure 6. The illustrated biography of the architect

4.3. Preliminary knowledge

The preliminary knowledge aims to acquire the necessary data to be able to start a preliminary modelling of the structure and to identify sources of uncertainty (aleatory and epistemic) that interest the case under examination.

For a correct identification of the existing structural system and its state of solicitation, it is important to reconstruct the process of realization and the following modifications made over time, as well as associated events.

Geometric-structural relief was referred both to the general geometry of the organism and to that of constructive elements, understanding the relationships with the possible structures in adherence; also the modifications made over time are represented in the relief.

The relief has allowed me to identify the resistant organism of the construction, the quality and the state of maintenance of the materials and the constitutive elements.

I have noticed the disarrangements, in action or stabilized, placing particular attention on the identification of the cracks and the mechanisms of damage.

Thanks to the collaboration with “Sibillina Dimora Srl” company and its staff, it was possible acquire important data regarding this case study.

These are just some data acquired during the technical inspection with the aim of acquiring the geometrical data necessary to structure modelling.

Figure 7. Geometric relief

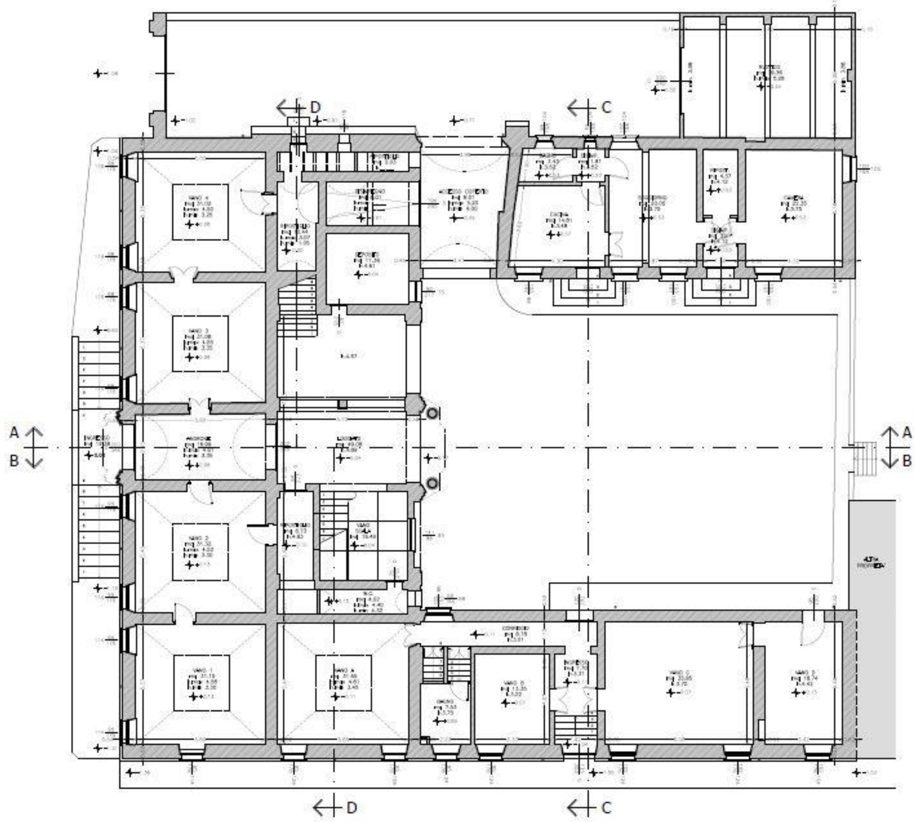


Figure 8. Palazzo Boldi – ground floor plan

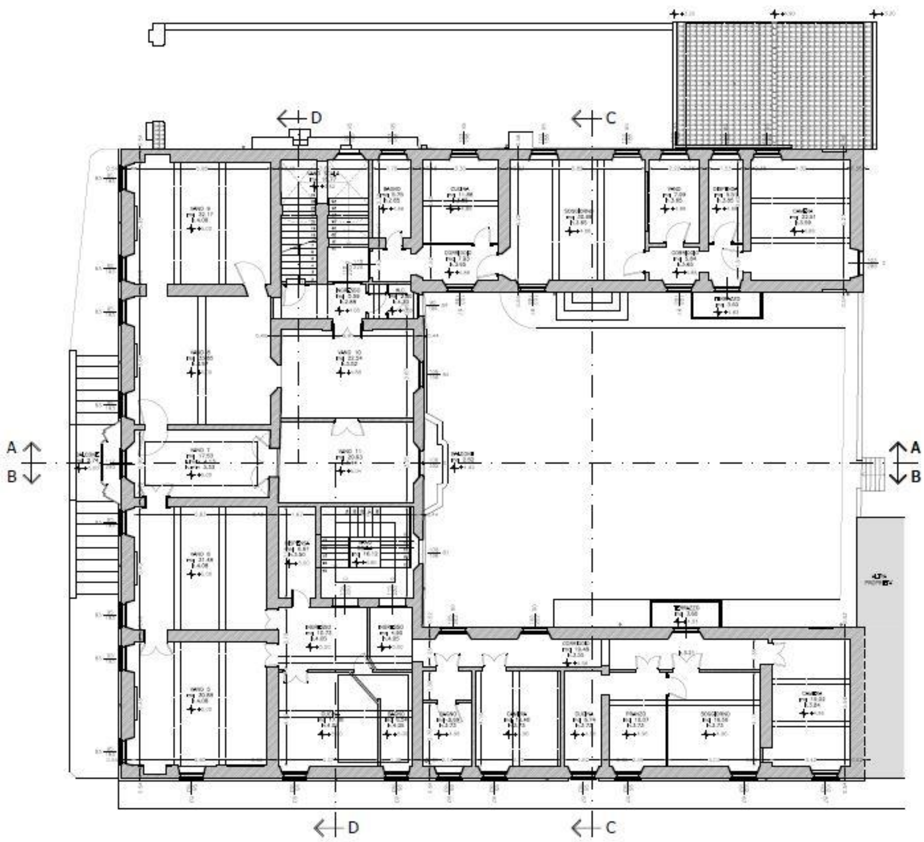


Figure 9. Palazzo Boldi – first floor plan

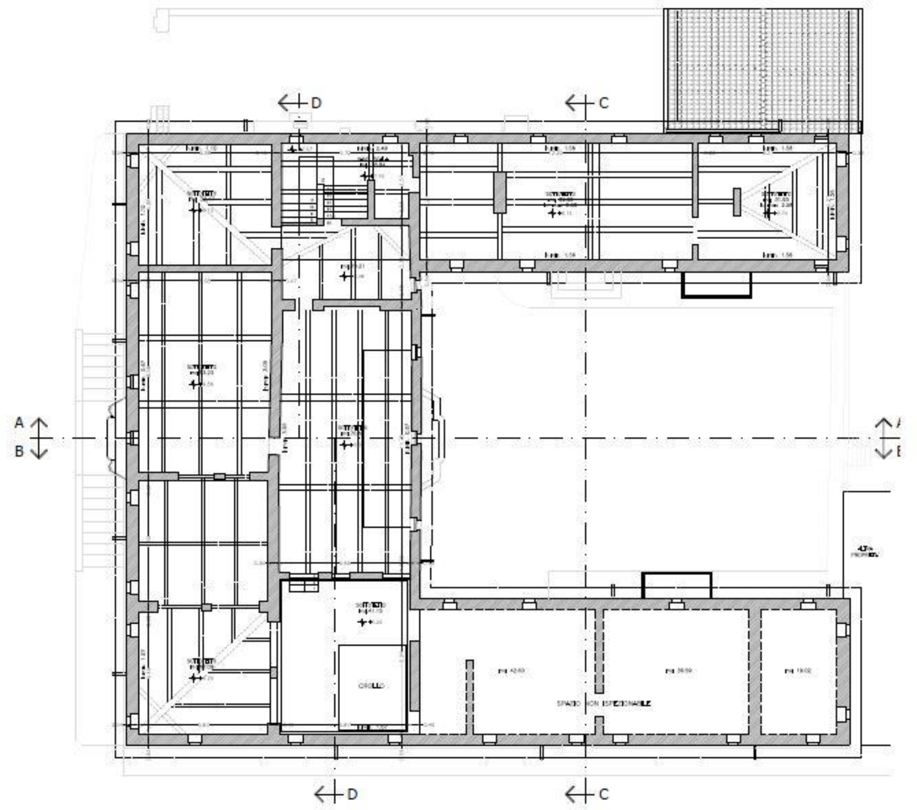


Figure 10. Palazzo Boldi – second floor plan

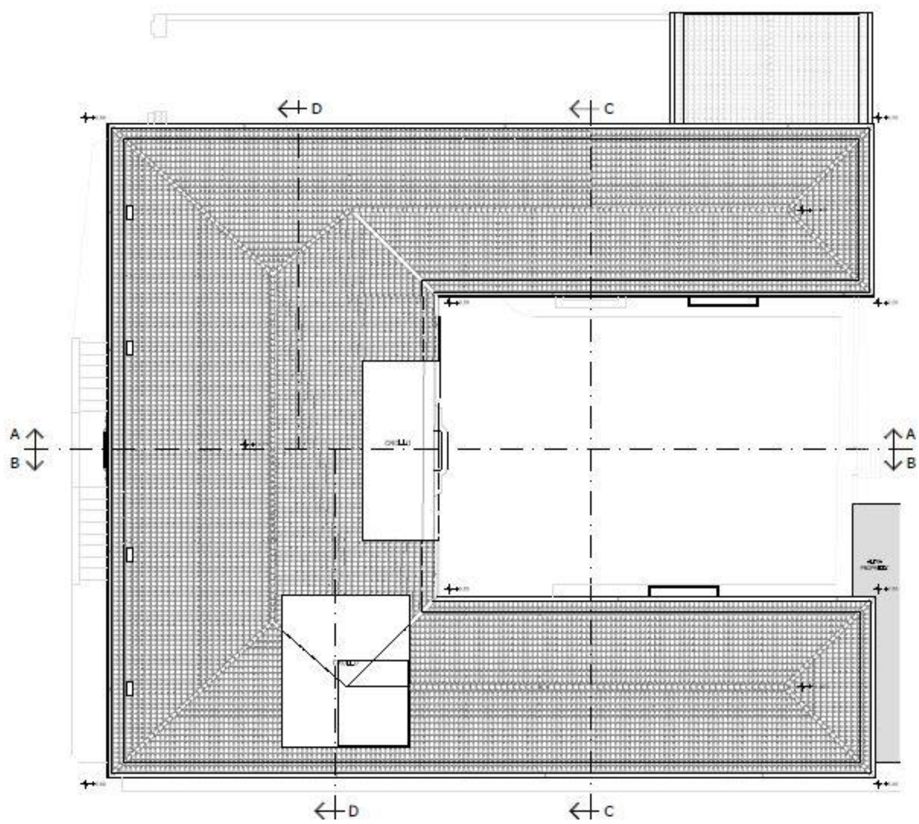


Figure 11. Palazzo Boldi – roof plan

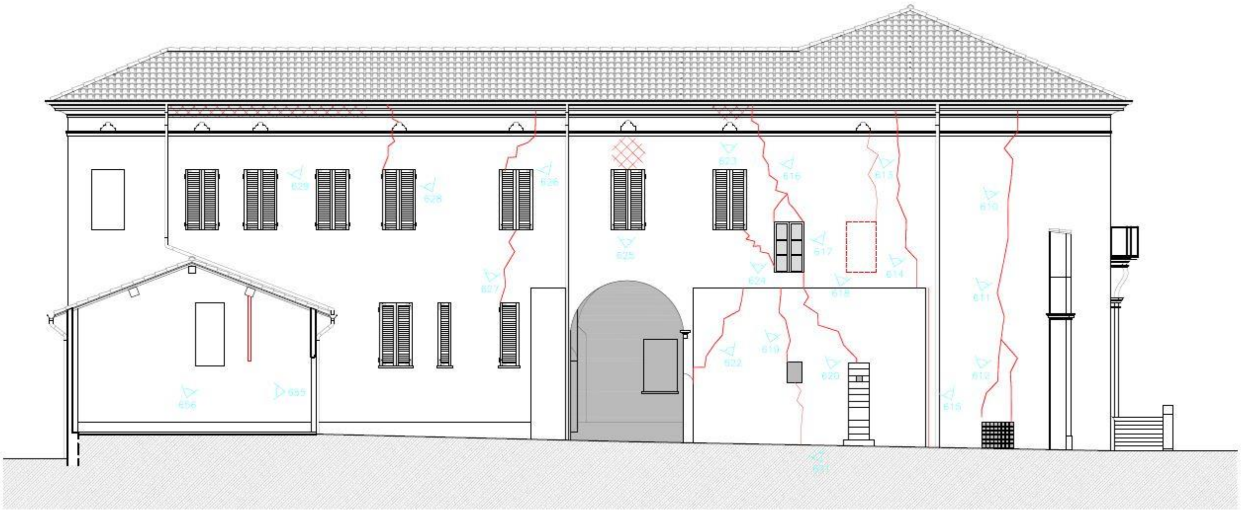


Figure 12. Palazzo Boldi – west elevation

Figure 13. Palazzo Boldi – south elevation

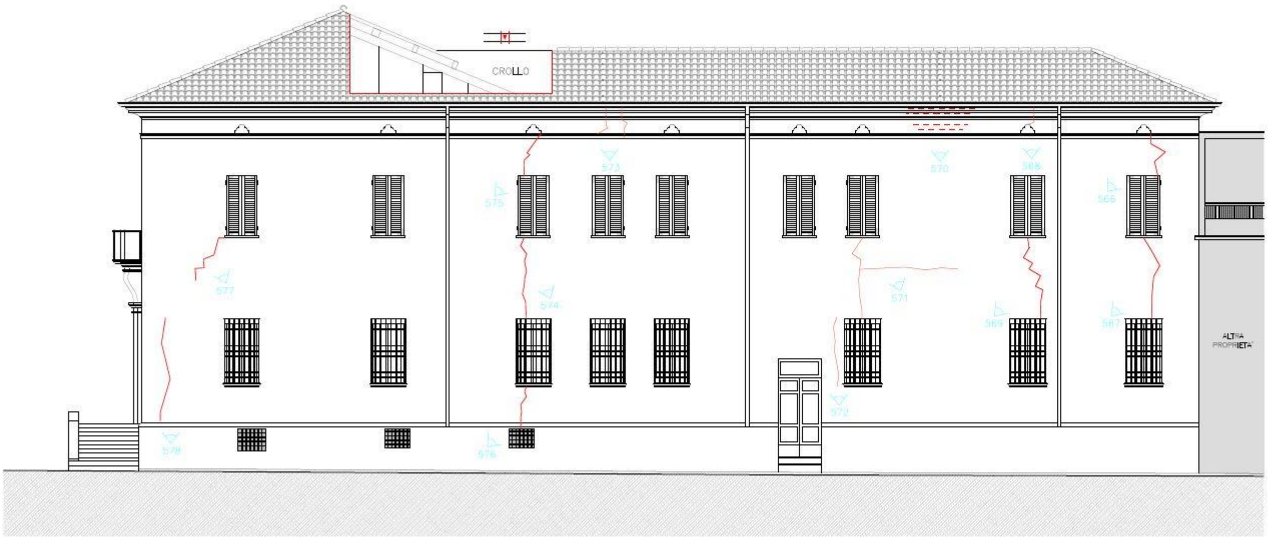


Figure 14. Palazzo Boldi – east elevation

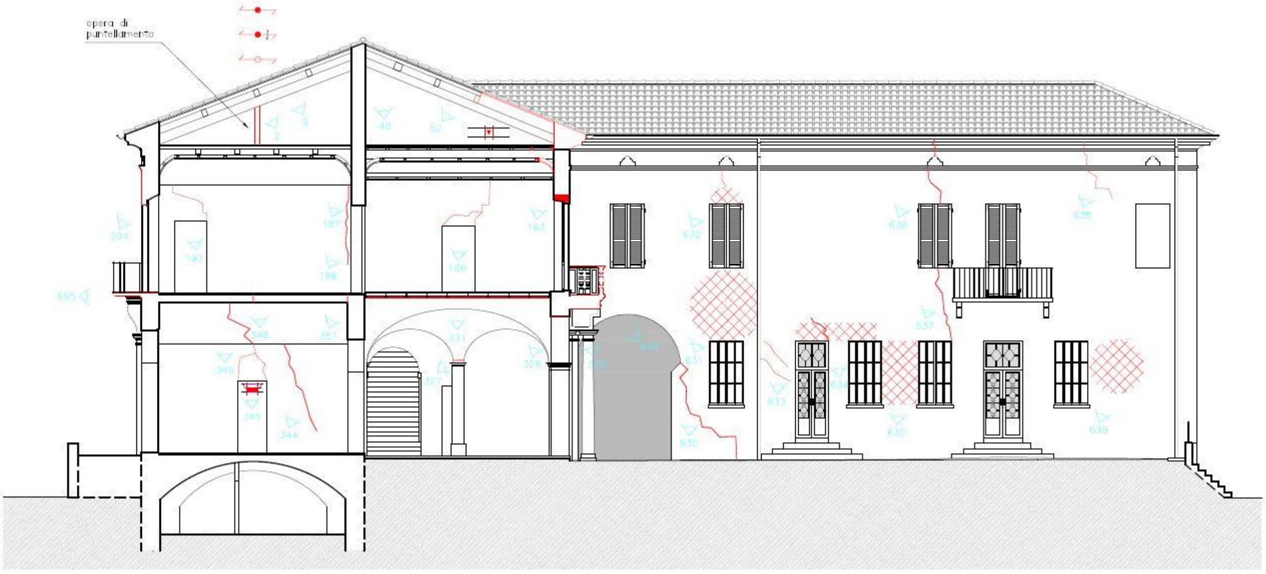


Figure 15. Palazzo Boldi – section A-A

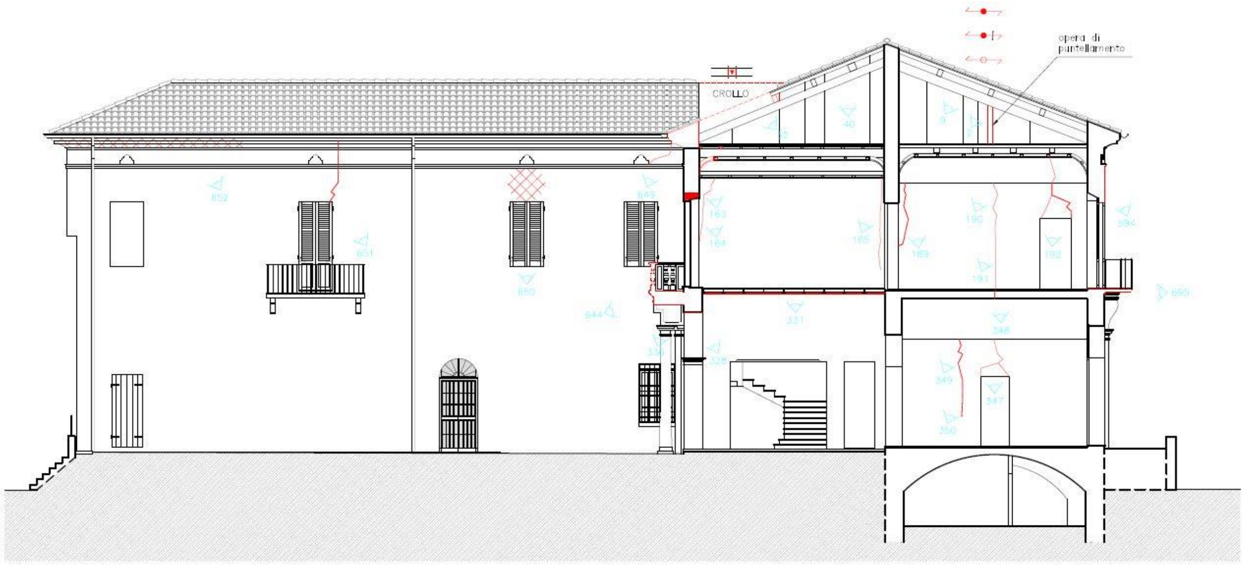


Figure 16. Palazzo Boldi – section B-B

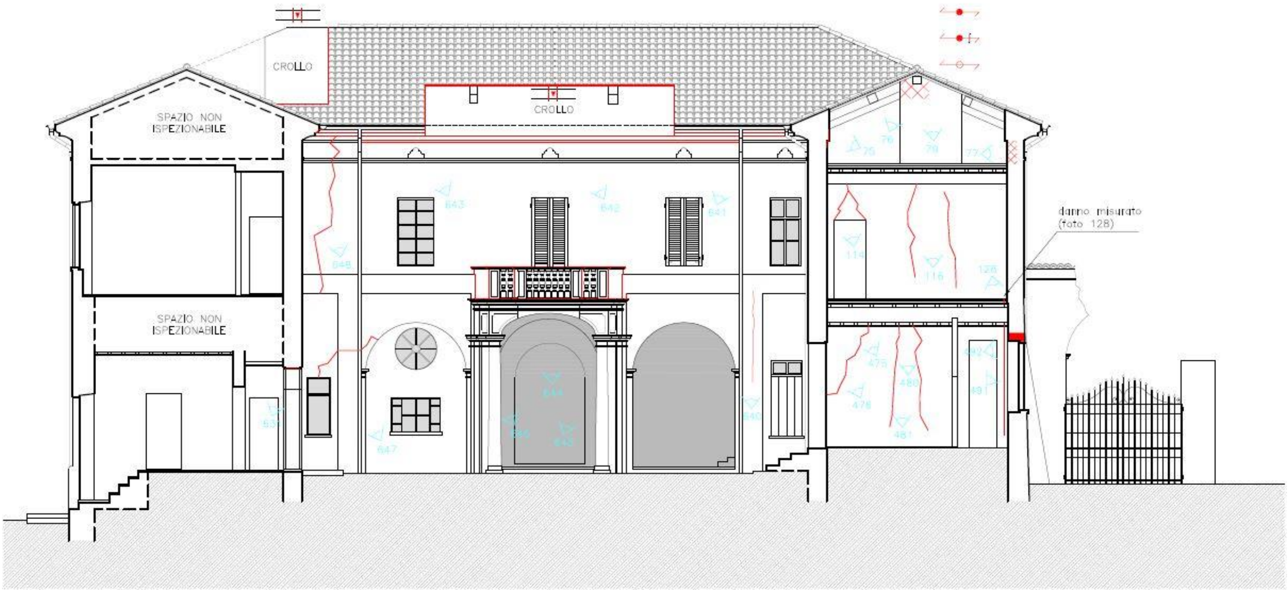


Figure 17. Palazzo Boldi – section C-C

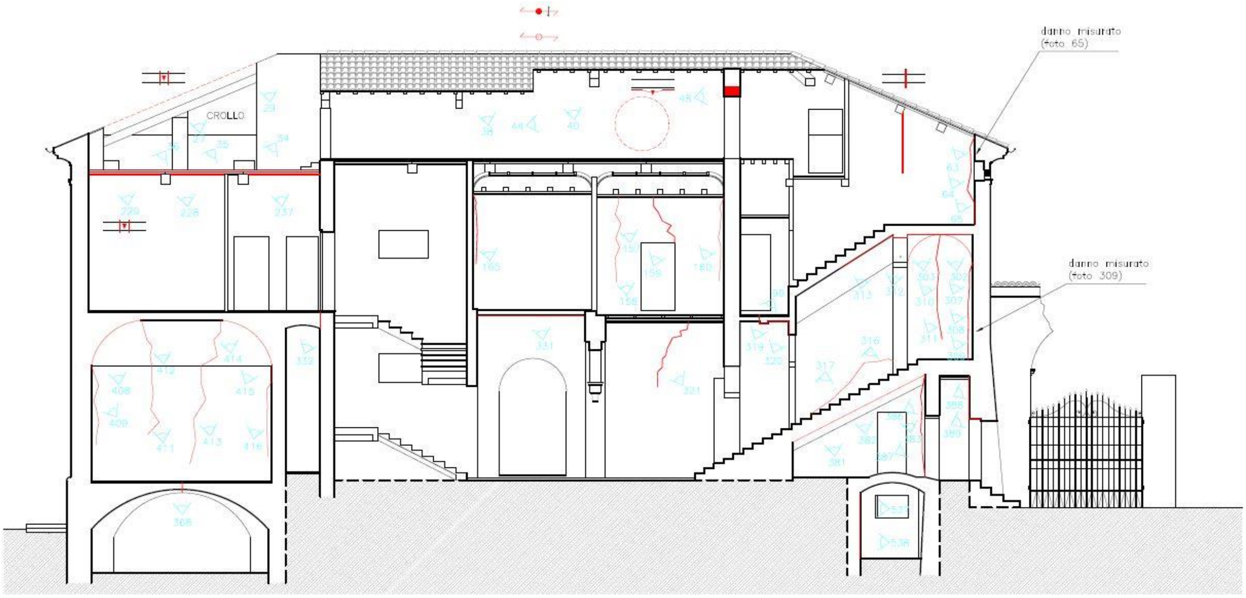


Figure 18. Palazzo Boldi – section D-D

4.4. Structure modelling

The code indications highlight the importance of carefully choosing the distribution of masses and rigidity (if necessary also considering the effect of non-structural elements) in order to obtain a structural model that is adequate for global analysis.

To that end, it is fundamental to conduct a preliminary knowledge phase, especially in the case of existing masonry structure, where the resistance structural system is not always immediately identifiable.

This can be due to structural variations or different construction phases, change in the type of use for the building, and modifications to the original plans.

The acquisition of this knowledge can make it clear what the resistant elements are (both for vertical actions as well as earthquake actions), as well as providing information about the characteristics of the materials.

A three-dimensional equivalent frame is the reference model, in which the walls are interconnected with horizontal partitions on the floors.

In the specific case of a masonry structure, the wall can be schematized as a frame, in which the resistant elements (piers and spandrel beams) and the rigid nodes are assembled.

The spandrel beams can be modelled only if they are adequately toothed by the walls, supported by structurally efficient architraves, and if possible a mechanism that is resistant to struts.

It is known that a less than perfect understanding of the positioning of the masses can lead to underestimation of the forces on the structures linked to torsional effects.

Indeed, the increasing eccentricity in the center of the masses and the center of rigidity is what exaggerates this aspect.

Hence, the code proposes consideration of accidental eccentricity to be applied to the center of the masses on every level of the structure.

Accidental eccentricity is equal to ±5% of the maximum dimension of the level considered by the building in a perpendicular direction to the seismic action.

The three-dimensional modelling used is the direct result of observation of real building behavior and experimental tests.

These allowed the introduction of some hypotheses about the structural behavior of masonry constructions.

As mentioned above, damage mechanisms observed in buildings can be divided into two categories.

These depend on the type of wall response and their mutual degree of connection: so-called first mode mechanisms, in which walls or portions of walls receive orthogonal forces on their plane; and second mode mechanisms in which the wall responds to the seismic action on its plane.

It is necessary to understand and identify the structure that is resistant to vertical and horizontal loads internal to the masonry construction to obtain a reliable simulation.

Usually, these elements are walls and horizontal structures.

Walls are assigned the role of resistant element, both with regards to horizontal and vertical loads.

The horizontal structures have the role of distributing the vertical load resting on them to the walls and then dividing, as part of the floors stiffening elements, the horizontal actions on the impacted walls.

With regards to the horizontal actions, the chosen model neglects the resistance contribution of the walls in orthogonal direction to their plane, given their notable flexibility.

Hence, the collapse mechanisms outside the plane are not modelled.

However, this is not a limitation as these are phenomena connected to the local response of the individual walls.

The onset of these can be decidedly limited by appropriate preventative actions.

Similarly, the flexional response of the planes is not simulated.

This is significant in checking their resistance, but can be ignored in terms of the global response. Loads on the plane are divided by the walls in function of the area of influence and warping direction.

The plane contributes as a slab with suitable level resistance.

The modelling was carried out by using the “Tremuri” program and adopting the piecewise linear constitutive laws for masonry panels; they allow the description of the non-linear response until very severe damage levels through progressing strength decay in correspondence of assigned values of drift, differentiating the behavior as a function of the main prevailing failure modes (if flexural, shear or mixed) and the element type (if pier of spandrel).

The preliminary model was defined based on the available data in the absence of specific diagnostic investigations.

The detailed knowledge phase, including a careful historical, architectural and technological analysis, allowed the choices of the preliminary mechanical properties adopted in the preliminary model.

- **INPUT PHASE: DEFINE GEOMETRY**

The definition of geometric parameters was supported by the results of a *in situ* geometric relief and a structural characterization of vertical elements, the soil type was also investigated.

In order to guarantee reliable structural modelling, the knowledge phase also regarded the identification of critical situations as the presence of flues, recesses, infill openings and not continuous walls, widespread in the buildings due to historical structural and functional changes. [33]

Figure 19. Floor type: timber diaphragms

Figure 20. Floor type: volts

The geometric data, mainly segments, are inserted directly in drawing mode, or by tracing a DXF or DWG file.

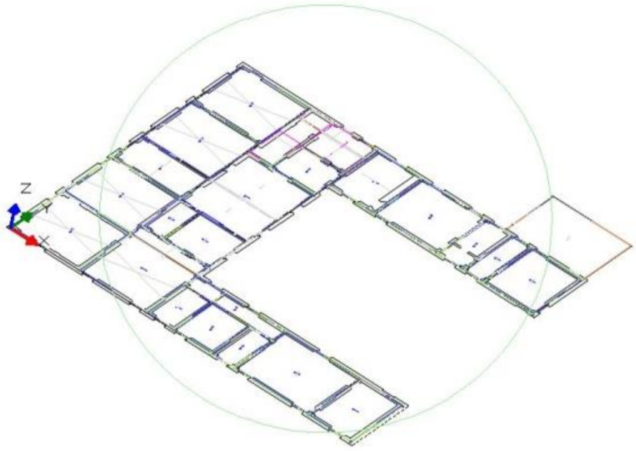


Figure 21. Import of dwg file by Tremuri program

The geometric characteristics of the structure, that is the placement of the walls in the plan and the height of the floors, constitute the foundation for insertion of the "structural objects" which constitute the resistant elements.

Dividing the wall into vertical areas which correspond to the various levels and noting the location of the openings, the portions of masonry, masonry piers, and spandrel beams where deformability and damage are concentrated, can be determined.

This can be verified by observing the damage caused by real earthquakes, and with experimental and numerical simulations.

These areas are modelled with finite two-dimensional macroelements, which represent masonry walls, with two nodes and three degrees of freedom per node *(ux, uz, roty)* and two additional internal degrees of freedom.

The resistant portions of the wall are considered as rigid two-dimensional nodes with finite dimensions, to which the macro-elements are connected.

The macro-elements transfer the actions along the level's three degrees of freedom, at each incident node.

In the description of each single wall, the nodes are identified by a pair of coordinates (*x, z*) in the level of the wall.

The height, z, corresponds to that of the horizontal structures.

The degrees of freedom are solely *ux, uz,* and *roty* (for two-dimensional nodes).

Thanks to the division of elements into nodes, the wall model becomes completely comparable to that of a frame plan.

During assembly of the wall, the possible eccentricities between the model nodes and the ends of the macro-elements are considered.

Given the axes that are the center of mass for the elements, these cannot coincide with the node. Hence in the rigid blocks, it is possible that eccentricity may be found between the model node and that of the flexible element.

During the insertion phase, I decided that each panel should go directly to the foundation in its low part so as to define the nodes.

Figure 22. The structural model: 3D view of the model

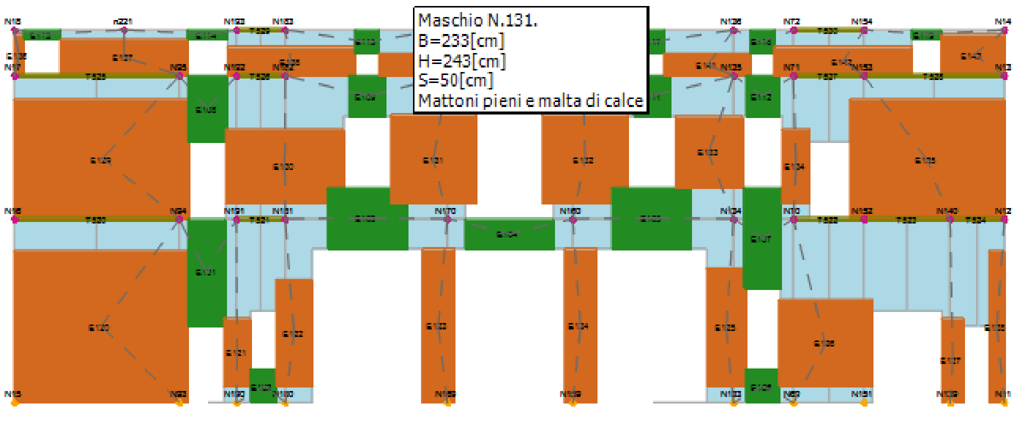


Figure 23. The structural model: equivalent frame idealization of one façade

Spandrel Pier Rigid node

- **INPUT PHASE: MASONRY STRUCTURAL CHARACTERISTICS**

The structure is composed of "structural objects" which constitute the resistant elements and each object is characterized by its material and additional geometric parameters (thickness, inertial characteristics, resistance properties).

Let us begin, for example, with all the vertical structures: they are generally characterized by solid brick and lime mortar masonry, the mechanical characteristics are shown in Figure 24.

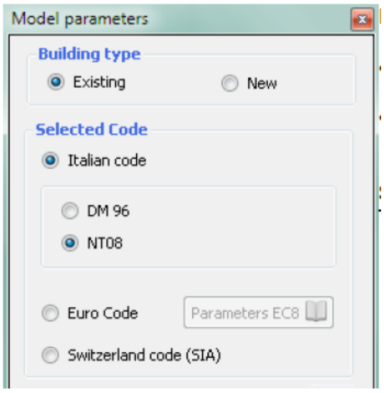


Figure 23. Model parameters

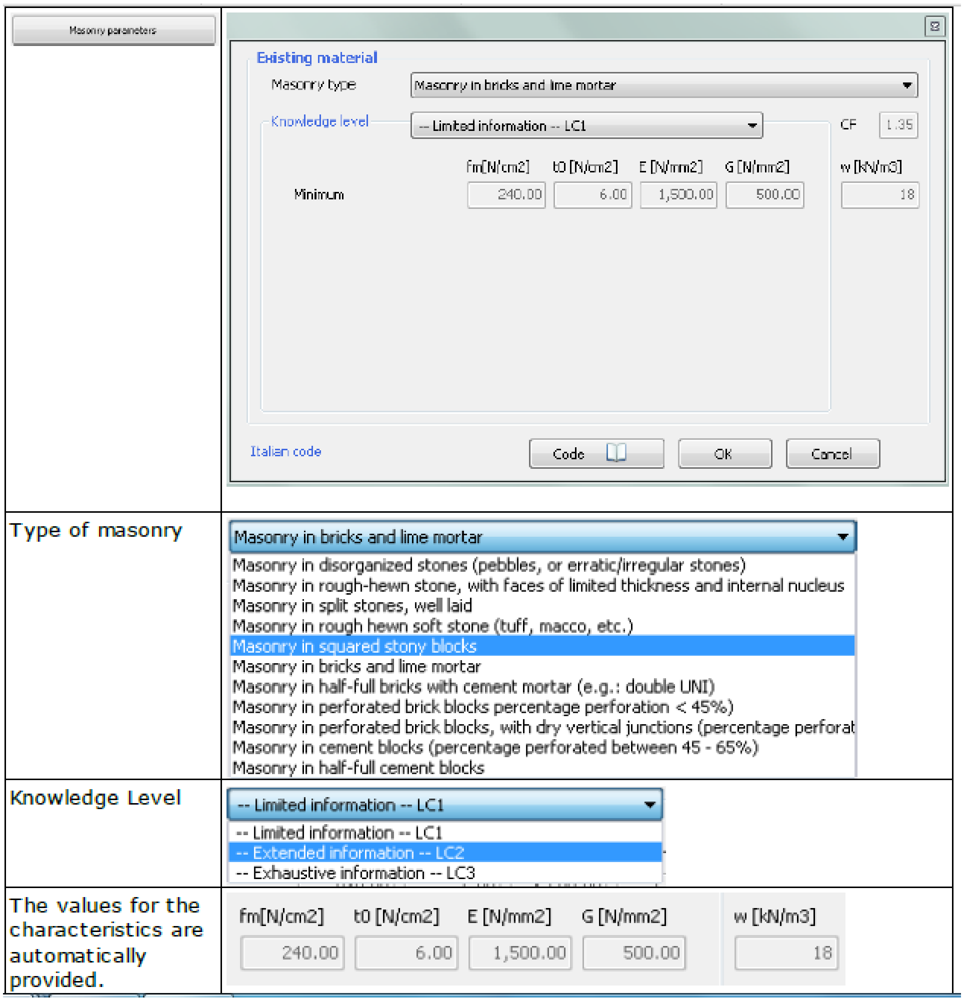


Figure 24. Masonry parameters

f_m: Average compressive strength

f_k: Characteristic compressive strength

t:*(Turnšek Cacovic)* Shear Strength

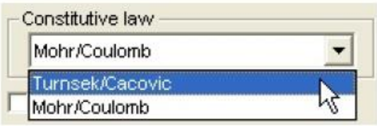
E: Longitudinal elasticity module

G: Shear elasticity module

w: Specific weight

CF: Confidence factor

The **Turnšek Cacovic** criteria represents a type of diagonal shear failure and its use is recommended especially for existing walls.



After having defined the material characteristics it is possible to define improvement parameters according to that which is indicated in the code, but I chose not to select any of them.

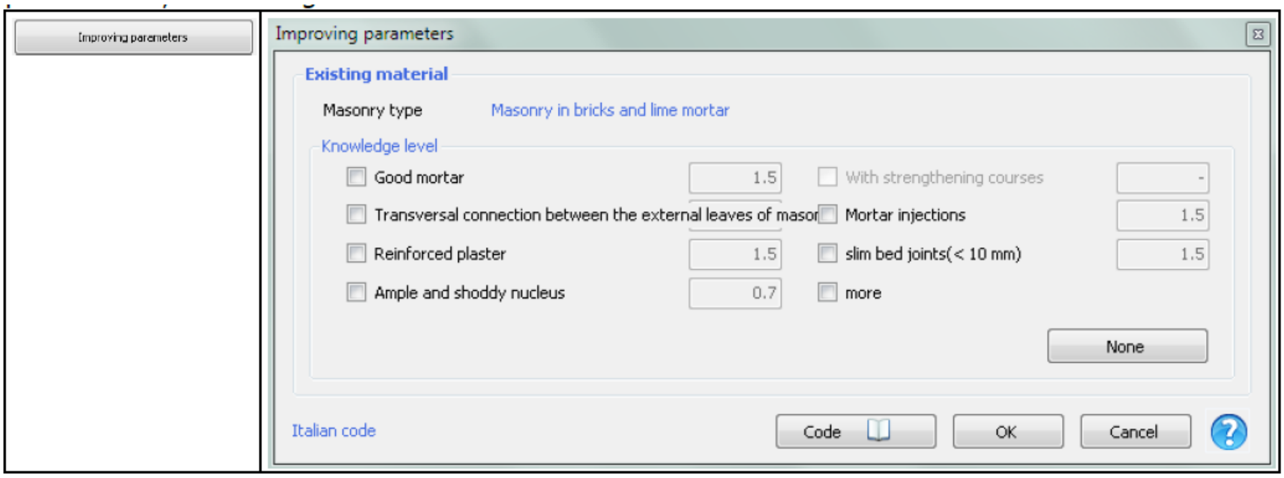


Figure 25. Improving parameters

- <u>**INPUT PHASE: SLABS, VAULTS AND ROOF STRUCTURAL CHARACTERISTICS**</u>

Starting from the 2D modelling of walls, the complete 3D model is obtained by also introducing horizontal elements: in particular, they are modelled as orthotropic membrane finite elements characterized by deformable stiffness.

This requirement is particularly relevant for those assets characterized by the presence of different kind of diaphragms: steel-beam and hollow flat block, wooden floors, vaulted floors.

Since generally the presence of deformable floors is such a crucial point in the knowledge of the seismic response of the preliminary model, the effects connected to a possible stiffening of the floors were considered.

Diaphragms (slabs, vaults, roof) transfer their vertical loads to the walls and divide the horizontal actions on the incident walls.

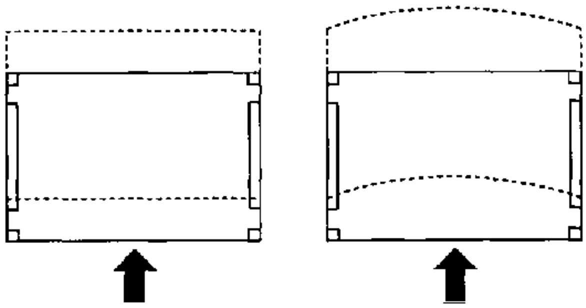


Figure 26. Rigid and deformable behavior of the diaphragm

The results of modal analyses obtained using deformable or rigid diaphragms will be compared later.

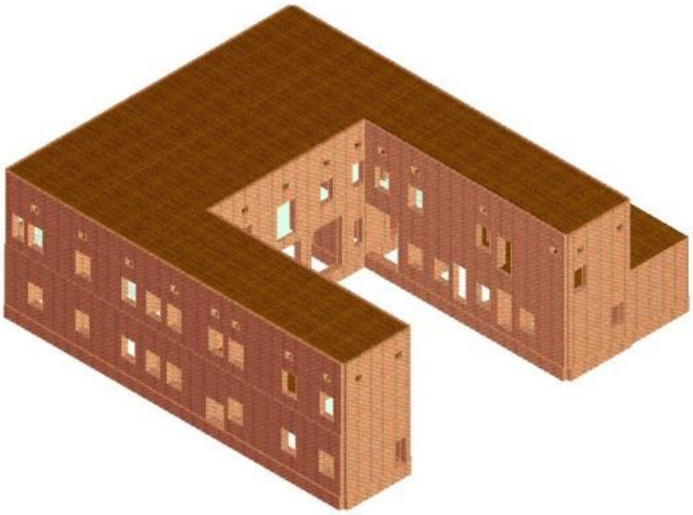


Figure 27. Horizontal elements

The load actions can be inserted on the slab as either permanent (Gk) or variable (Qk), that can be combined according to the coefficients indicated in the code.

The **permanent loads (G_k)** are defined as permanent structural loads (G1).

The **permanent loads ($G_{k,agg}$)** are defined as the weight of all non-structural elements (G2).

It is necessary to check that the slab being examined is covered and indicate the support length of the floor on the wall.

Figured 28. Different kind of slabs

The horizontal elements window allows us to set the mechanical characteristics and the warping direction of various type of slabs among the most common, I have selected the following:

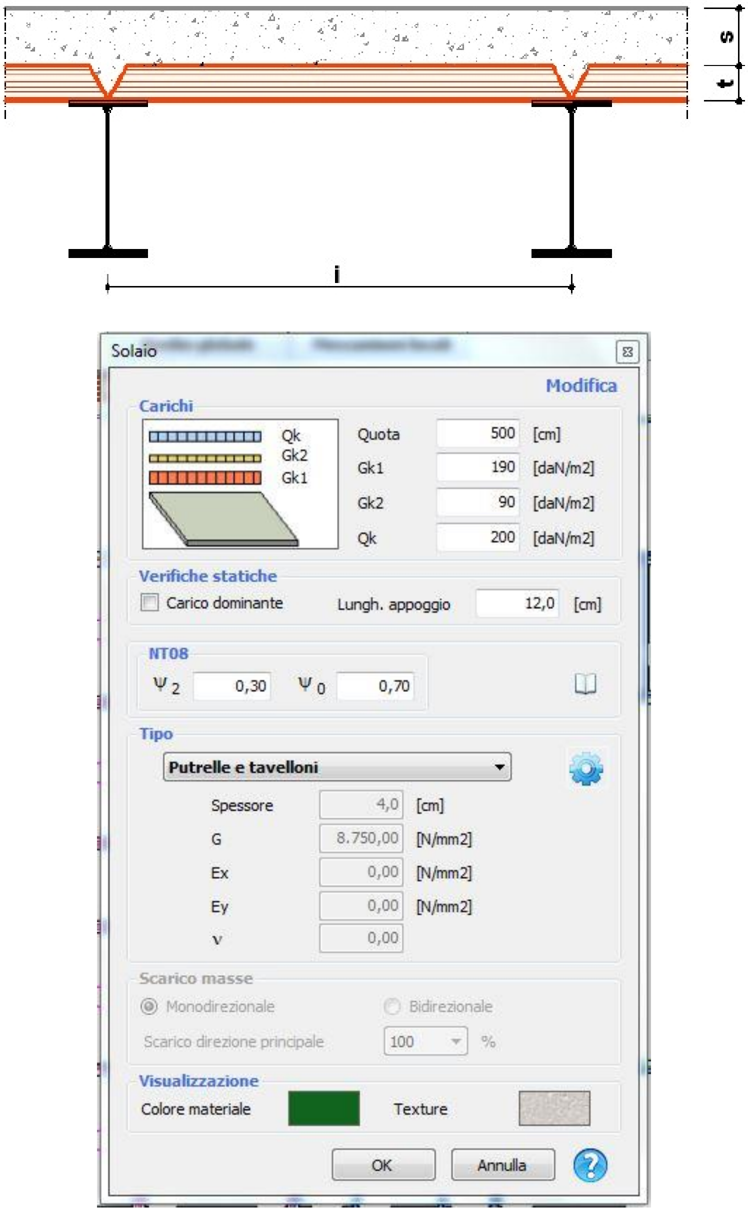


Figure 29. Horizontal structure: steel-beam and hollow flat block

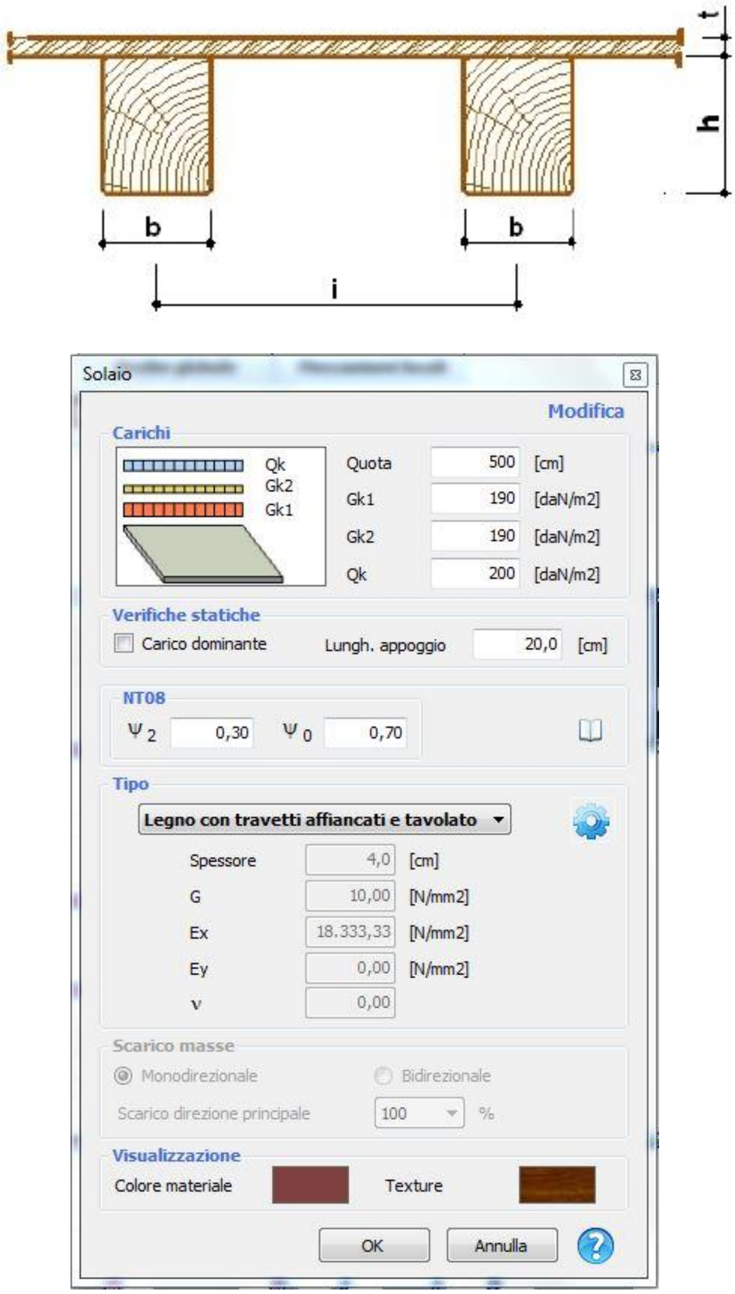


Figure 30. One way timber floor with single wood plank

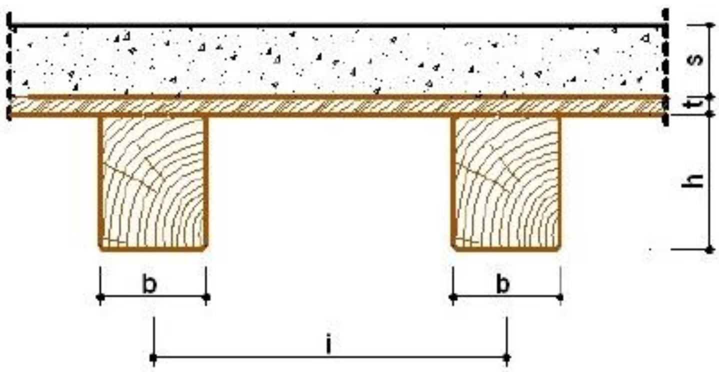


Figure 31. One-way timber floor with additional concrete topping

It’s also possible to insert the desired vault type.

Figured 32. Different kind of vaults

For each vault typology I inserted the mechanical characteristics and the direction for the vault’s discharge.

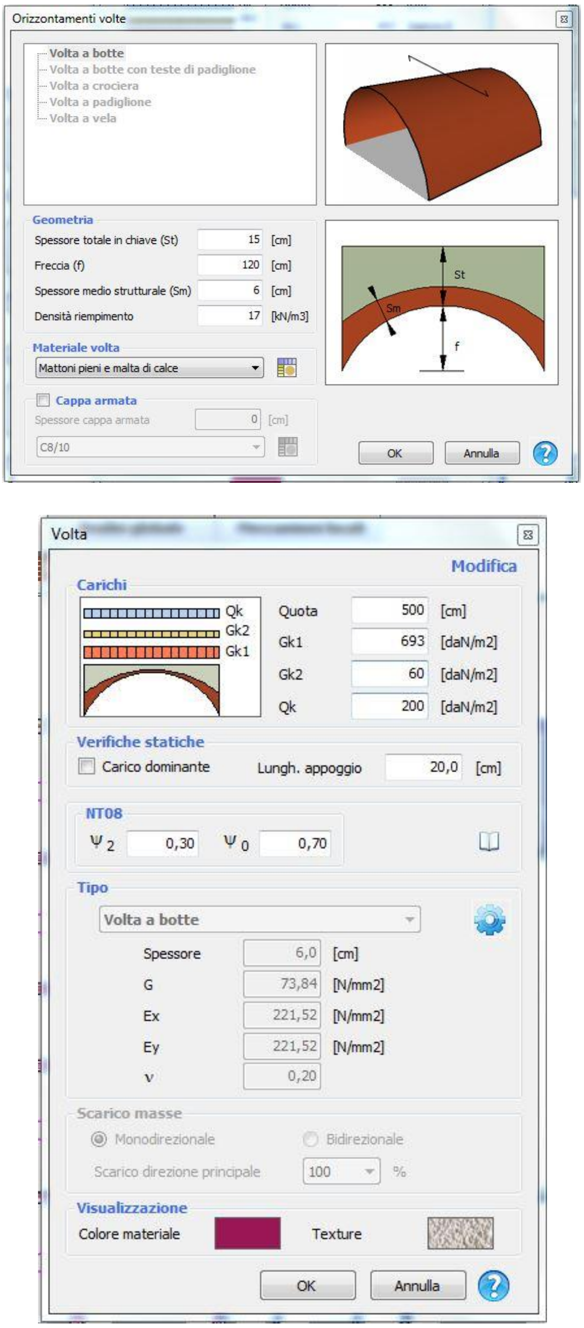


Figure 33. Barrel vault

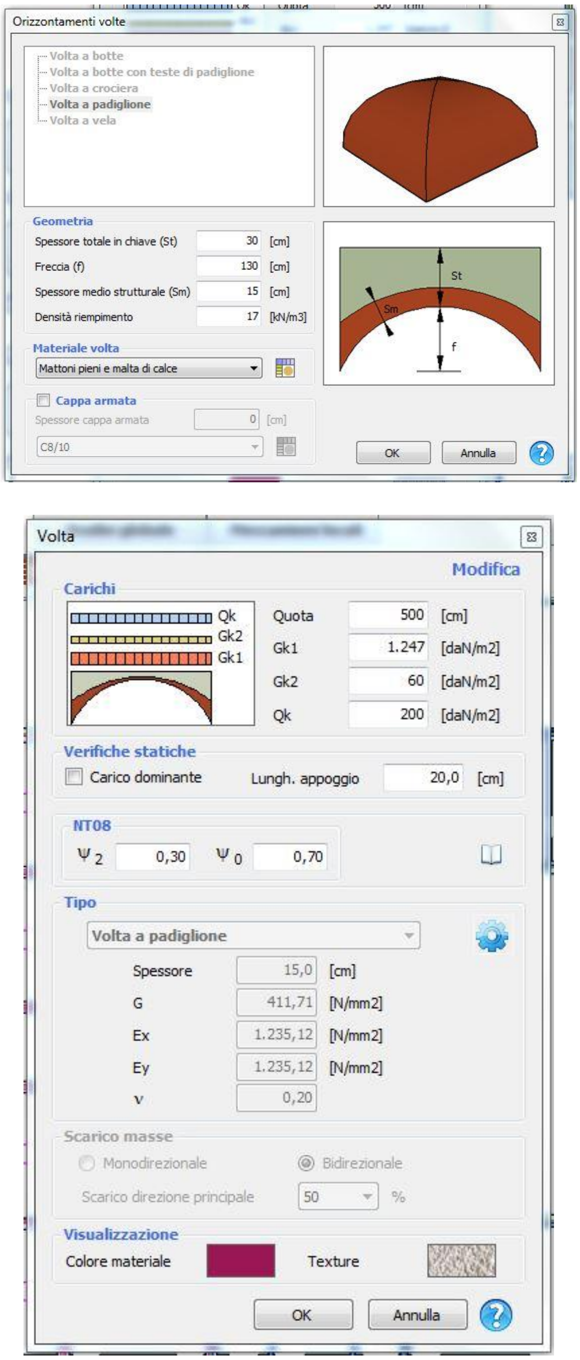


Figure 34. Cloister vault

Using the “Tremuri” model, the data for the equivalent frame are derived based on the geometry and the inserted structural objects.

After the analysis a mesh is created which schematizes piers, spandrel beams, beams, tie-beams, and columns; these elements can also be manually modified.

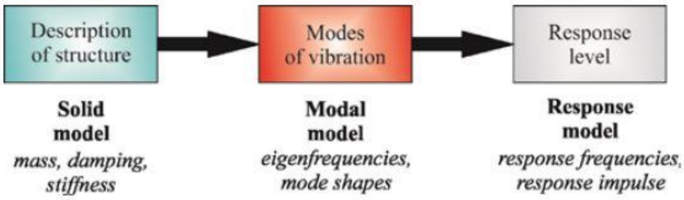
4.5. Modal analysis: deformable diaphragms versus rigid diaphragms

Using modal analysis we can determine natural frequencies, damping factor, and modal forms.

The importance of the evaluation of the modal parameters can be crucial for a better understanding of the building's structural behavior and for reducing the uncertainties in the assessment of the actual vulnerability.

The Figure shows the theoretical procedure of vibration analysis, in particular describing the three-phase procedure of theoretical vibration analysis.

Let's start with a description of the physical properties of the structures: mass, stiffness and damping characteristics. [30]



The theoretical modal analysis of solid model leads to a description of the behavior of the structure as modes of vibration, a so-called modal model.

This model is defined as a set of natural frequencies with damping factor and natural modes vibration.

The response of the model is the next part of the theoretical procedure analysis, which corresponds to the excitation and its amplitude.

The model describes a set of frequency response functions. [30]

The theory is described in Pavol Lengvarsky, Jozef Bocko, "Theoretical Basis of Modal Analysis", Department of Applied Mechanics and Mechatronics, Technical University of Košice, Košice, Slovakia - American Journal of Mechanical Engineering, 2013, Vol. 1, No. 7, 173-179.

Now it is possible to compare modal analyses results obtained using deformable or rigid diaphragms.

This is the results of modal analysis obtained <u>modelling horizontal structures as **deformable bodies**</u>:

Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0,77009	0	0,00	109.575	3,84	0	0,00
2	0,36528	17	0,00	16.247	0,57	4	0,00
3	0,33256	2.570	0,09	228.397	8,00	38	0,00

Figure 35. List of modal forms – *<u>DEFORMABLE PLANES</u>*

The table shows the main results of the modal analyses in terms of percentage of mass participation (%Mx, %My and %Mz) and period (T), with which is possible to derive the frequency (n).

The results were illustrated by selecting the first 3 modes.

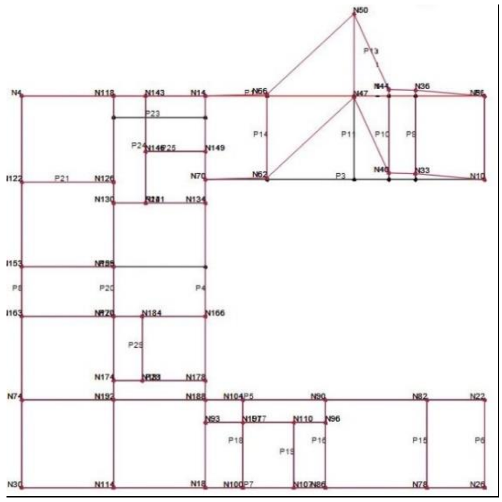


Figure 36. _DEFORMABLE PLANE_ - Mode 1 – Frequency(Hz)=1.30

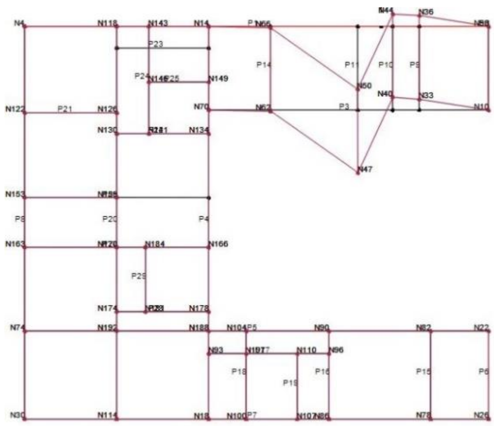


Figure 37. _DEFORMABLE PLANE_ - Mode 2 – Frequency(Hz)=2.74

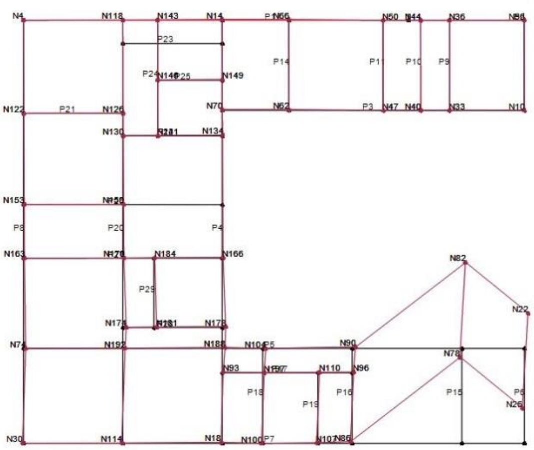
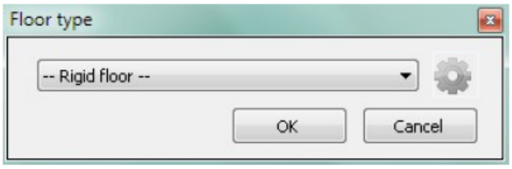
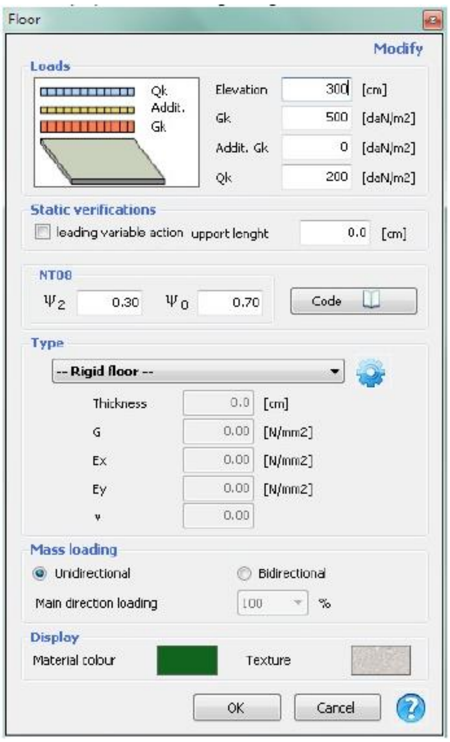


Figure 38. _DEFORMABLE PLANE_ - Mode 3 – Frequency(Hz)=3.01

As can be seen from the fundamental forms and periods, the use of deformable bodies has determined an irregular behavior due to a worse distribution of the actions through the horizontal diaphragms and a worse global response of the building.

<u>Modelling horizontal structures as **rigid bodies**</u>, however, the following results were obtained:





Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0,28264	19.332	0,68	1.899.657	66,50	35	0,00
2	0,24878	150.071	5,25	94.619	3,31	11	0,00
3	0,23882	2.068.052	72,40	61.326	2,15	41	0,00

Figure 39. List of modal forms – *<u>RIGID PLANES</u>*

The table shows the main results of the modal analyses in terms of percentage of mass participation (%Mx, %My and %Mz) and period (T), with which it's possible to derive the frequency (n). The results were illustrated by selecting the first 3 modes.

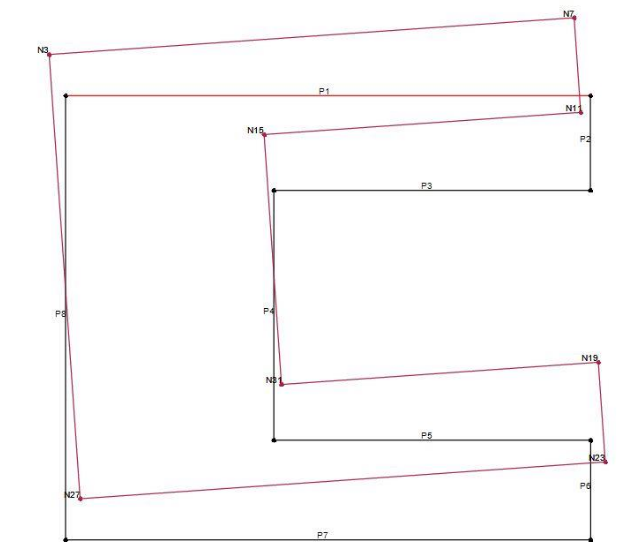


Figure 40. *RIGID PLANE* - Mode 1 – Frequency(Hz)=3.54_Transversal

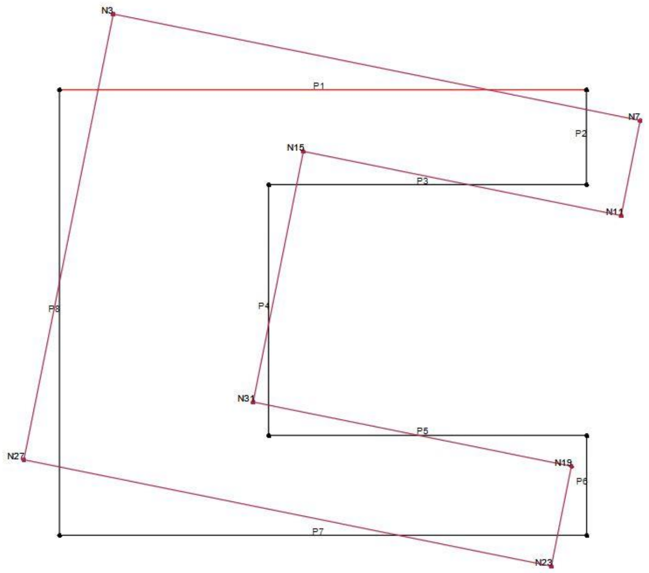


Figure 41. *RIGID PLANE* - Mode 2 – Frequency(Hz)=4.02_Torsional

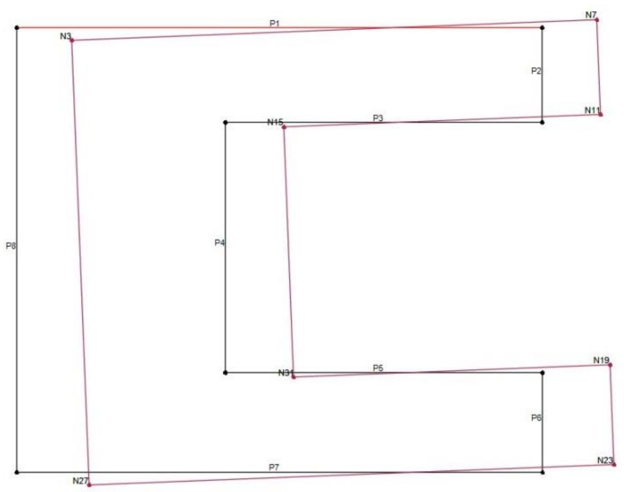


Figure 42. *RIGID PLANE* - Mode 3 – Frequency(Hz)=4.19_Longitudinal

As can be seen from the fundamental forms and periods, the use of rigid bodies has determined a more regular whole behavior, promoting a better distribution of the actions through the horizontal diaphragms and a better global response of the building.

Indeed, from the modal analysis results it was possible to deduce that the first vibration mode is transversal, the second vibration mode is torsional, while the third mode is longitudinal.

Furthermore, the periods of vibration and the participating masses change significantly by ensuring that a rather significant mass participation is involved.

By comparing theise results obtained by modal analysis on preliminary model with the results obtained by “OMA” I will be able to improve the knowledge of fem model of “Palazzo Boldi” to use in the sensitivity analysis.

4.6. Calibration of the preliminary model through Operational modal analysis (OMA)

Through Operational Modal Analysis it is possible to limit the uncertainty due to modelling errors on the preliminary model.

The purpose was to set the elastic parameters that characterize the linear response under low energy inputs.

Thanks to the collaboration with the Indaco Srl Company, specializing in diagnostic investigation of existing buildings, I had the possibility to perform some dynamic measures on the structure.

Information about the global response of the building such as modal forms and periods of oscillation were measured.

GENERAL TECHNICAL DATA

NUMBER OF CHANNELS	8 or 16 simultaneous channels
MEASURABLE SIGNALS	Tension, Full bridge (IEPE, Charge, Thermocouples e RTD /MSI adapter
RESOLUTION	24 bit
ADC TYPE	Sigma - Delta
ACQUISITION FREQUENCY	10 to 200 000 Hz 204.8 kS/s
INPUT RANGE	Voltage ±0,01 V, ±0,1 V, ±1 V, ±10 V; Voltage via MSI adapter; Full bridge ±10 mV/V, ±100 mV/V, ±1000 mV/V; Half bridge ; IEPE—MSI adapter ±0,1 V, ±1 V, ±10; Thermocouples (-200°C to 1000°C and from 0 to 6.5kOhm)
DC ACCURACY	10V range: 0,1% of value, +1 mV 1 V range: 0,1% of value, +0,5 mV 100mV range: 0,1 of value, +0.1 mV 10mV range: 0.1% of value, +0.1 mV
TENSION SENSORS	±5 V 0.1% @ 100mA, 12V@400mA for single channel
OVERLOAD PROTECTION	±0 70V

Figure 43-a. 8-channels DaTa500 acquisition system

Figure 43-b. Piezoelectric accelerometer - KS48C

		KS48C	
Output		IEPE	
Piezo design		Shear	
Voltage Sensitivity	B_{ua}	10 000±5% (1)	mV/g
Range	a_+/a_-	± 6	g
Destruction limit	a_{max}	1000	g
Linear Frequency Range	$f_{3\,dB}$ $f_{10\%}$ $f_{5\%}$	0,1 .. 4000 0,2 .. 2600 0,3 .. 2000	Hz Hz Hz
Resonant frequency	f_r	> 7 (+25 dB)	kHz
Operating temperature range	T_{min}/T_{max}	-20 / 120	°C
Weight without cable	m	165 / 5.8	g / oz
Case material		Edelst.• Stainl. St	
Cable connection		Axial	
Socket		Binder 713	
Mounting thread		M8	

Figure 43-c. Technical specification piezoelectric accelerometer - KS48C

All ambient vibration tests were carried using a 8-channel data acquisition system 24-bit resolution and 4 high sensitivity mono-axial seismic accelerometers for acquiring low frequency vibration signals.

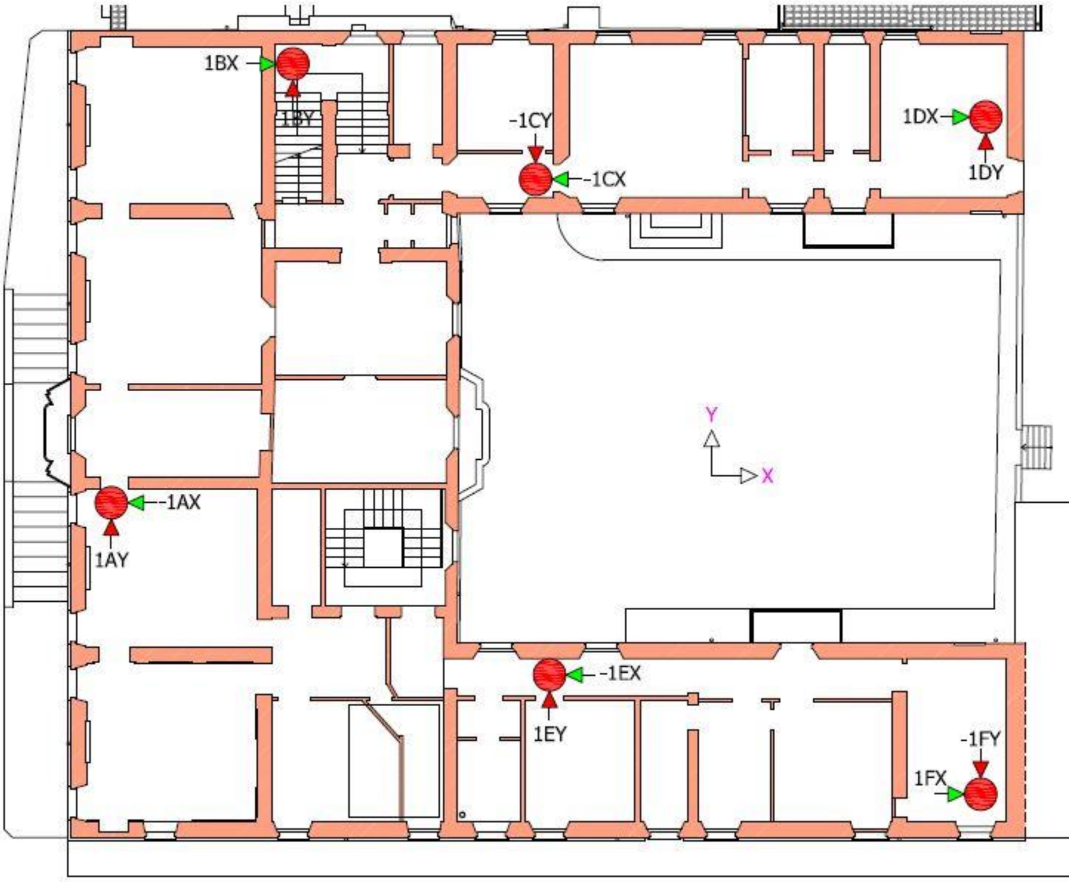
These sensors are piezoelectric units and their high sensitivity enables them to be used for dynamically analyzing and monitoring buildings subject to continuous vibration.

The experimental characterization of the dynamic parameters of the structure was operated by exploiting only environmental excitement.

The dynamic response of the structure was measured in correspondence of the first floor level for a total of six measuring points.

Every position of measure was installed with two mono-axial accelerometers parallelly directed to the principal directions of the building.

The six effected recordings lasted about 30 minutes and all this was possible by using an advanced data acquisition and recording software:” Dewesoft X3”.



	PIEZOELECTRIC ACCELEROMETER dir.X-Y
1CX-1CY 1DX-1DY	TEST N.1
1CX-1CY 1BX-1BY	TEST N.2
1CX-1CY 1EX-1EY	TEST N.3
1CX-1CY 1FX-1FY	TEST N.5
1CX-1CY 1AX-1AY	TEST N.6

Figure 44. Layout of the dynamic monitoring system - first floor plan

Figure 45. DaTa500 acquisition system

Figure 46. DaTa500 acquisition system

Figure 47. Piezoelectric accelerometer

Figure 48. Piezoelectric accelerometer

Figure 49. Piezoelectric accelerometer

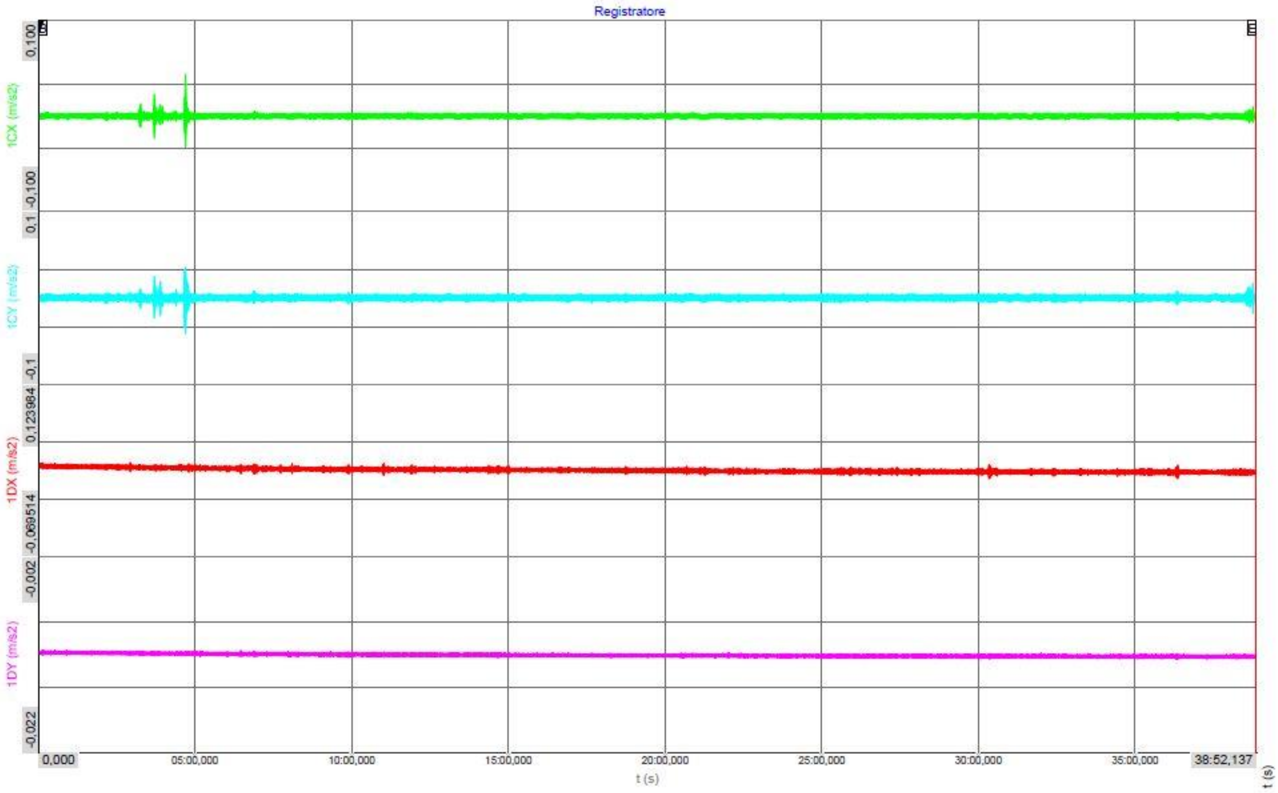


Figure 50. Recording 1

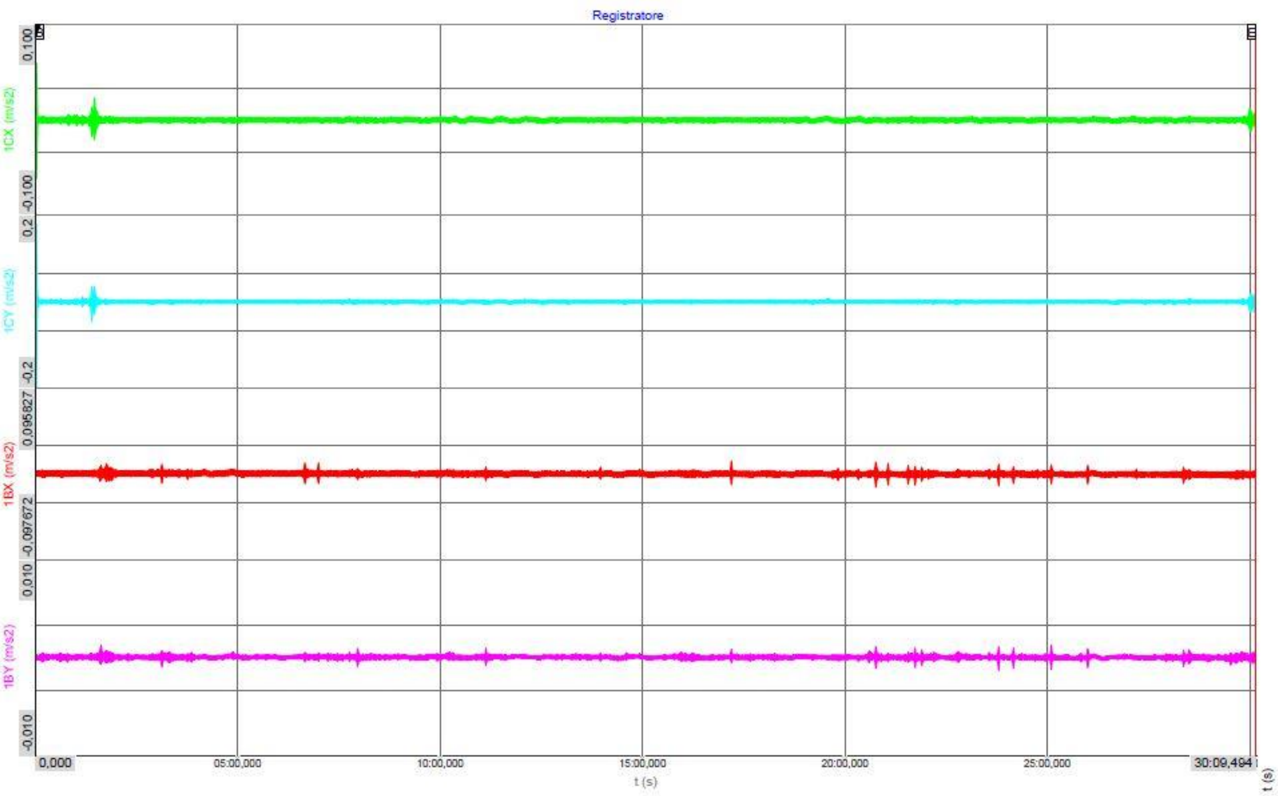


Figure 51. Recording 2

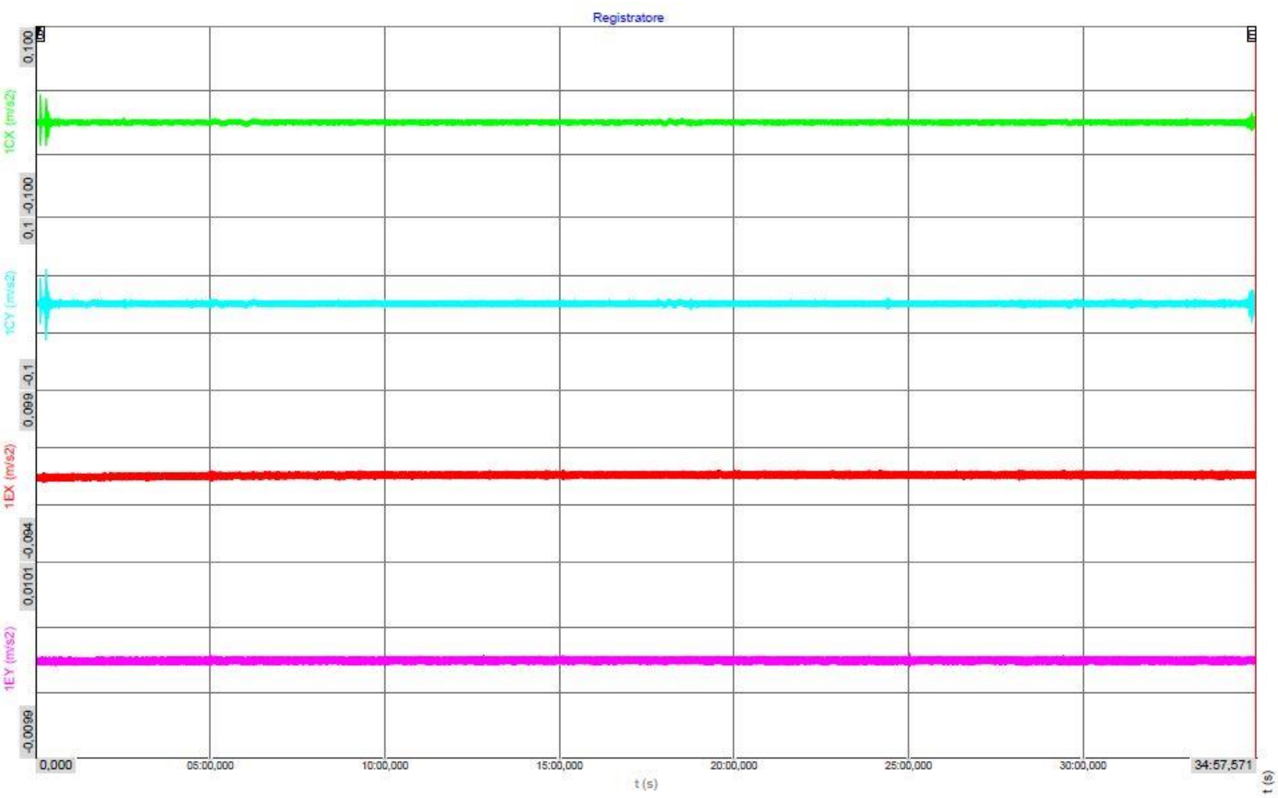


Figure 52. Recording 3

To perform Operational Modal Analysis it was necessary to export the acquired data into a powerful software able to supply modal frequencies, modal shapes and modal damping: “Artemis Modal Pro”.

First of all, it was necessary to insert the geometry of the structure through points, lines and surfaces; subsequently, the node equations for rigid body motions were defined.

In this case the measurements of the geometry building model was divided into five test setups.

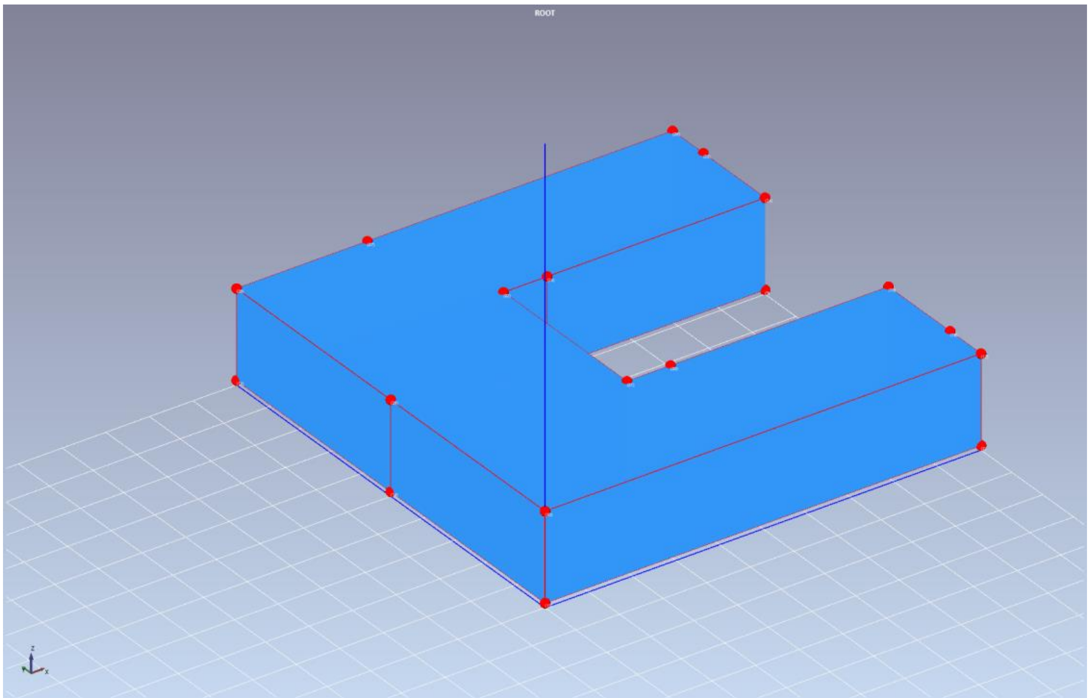


Figure 53. Definition of the geometry

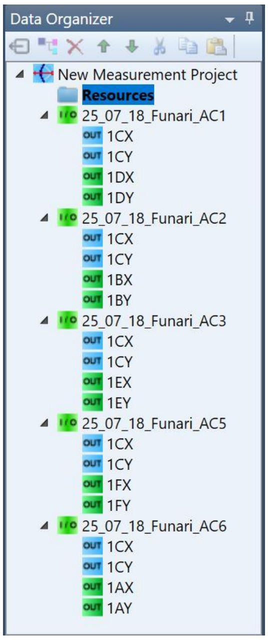


Figure 54. Importing measurements

Before assign DOF information it was necessary to upload the previously measured data.

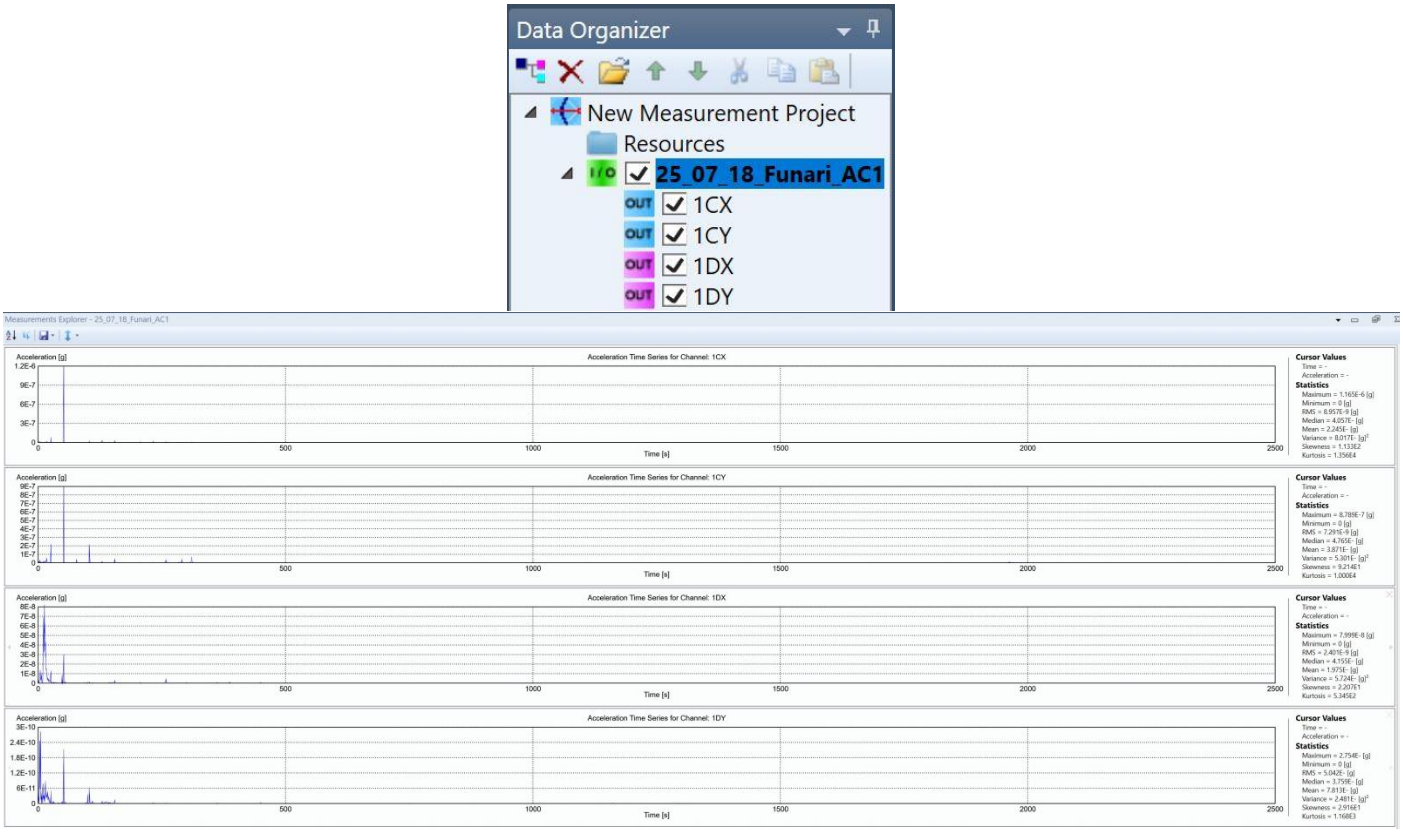


Figure 55. Test setup item 1

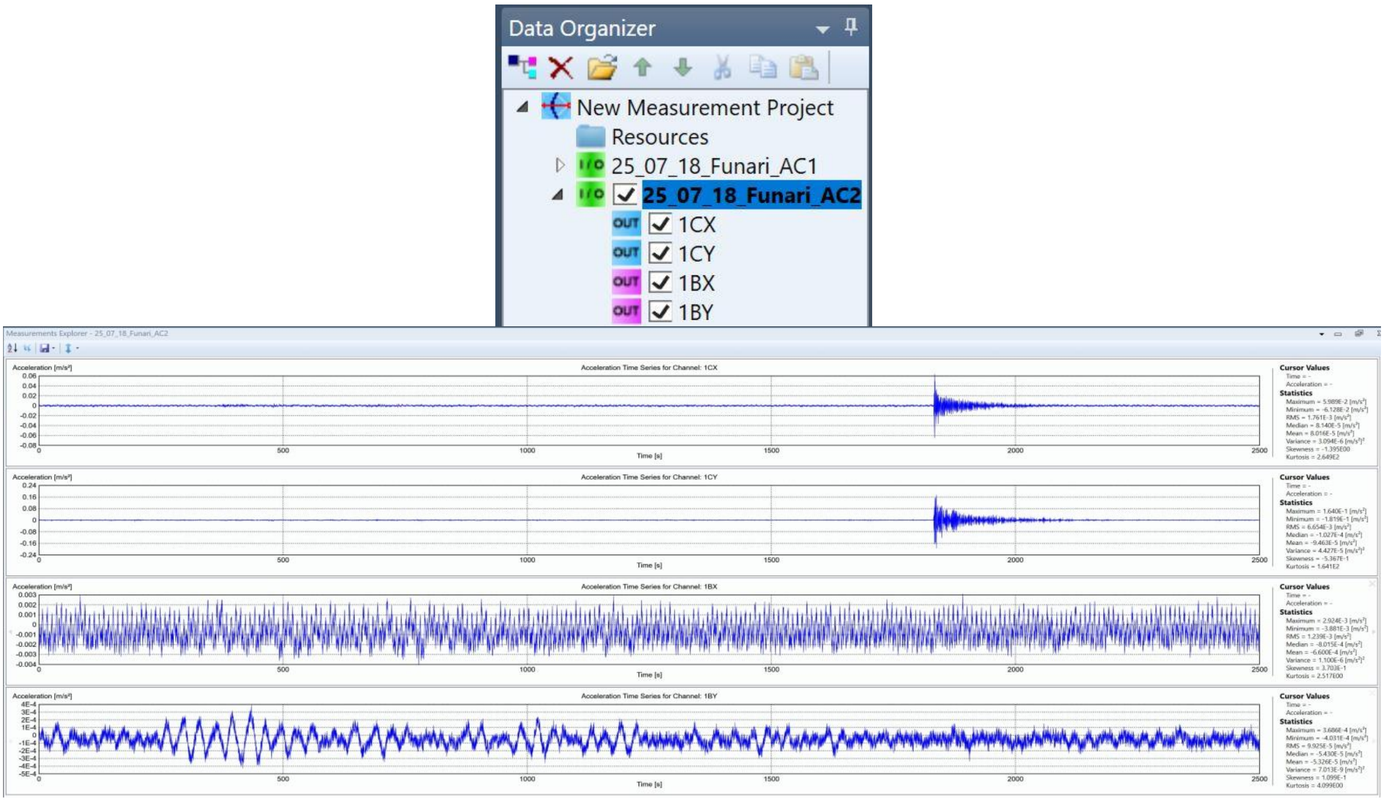


Figure 56. Test setup item 2

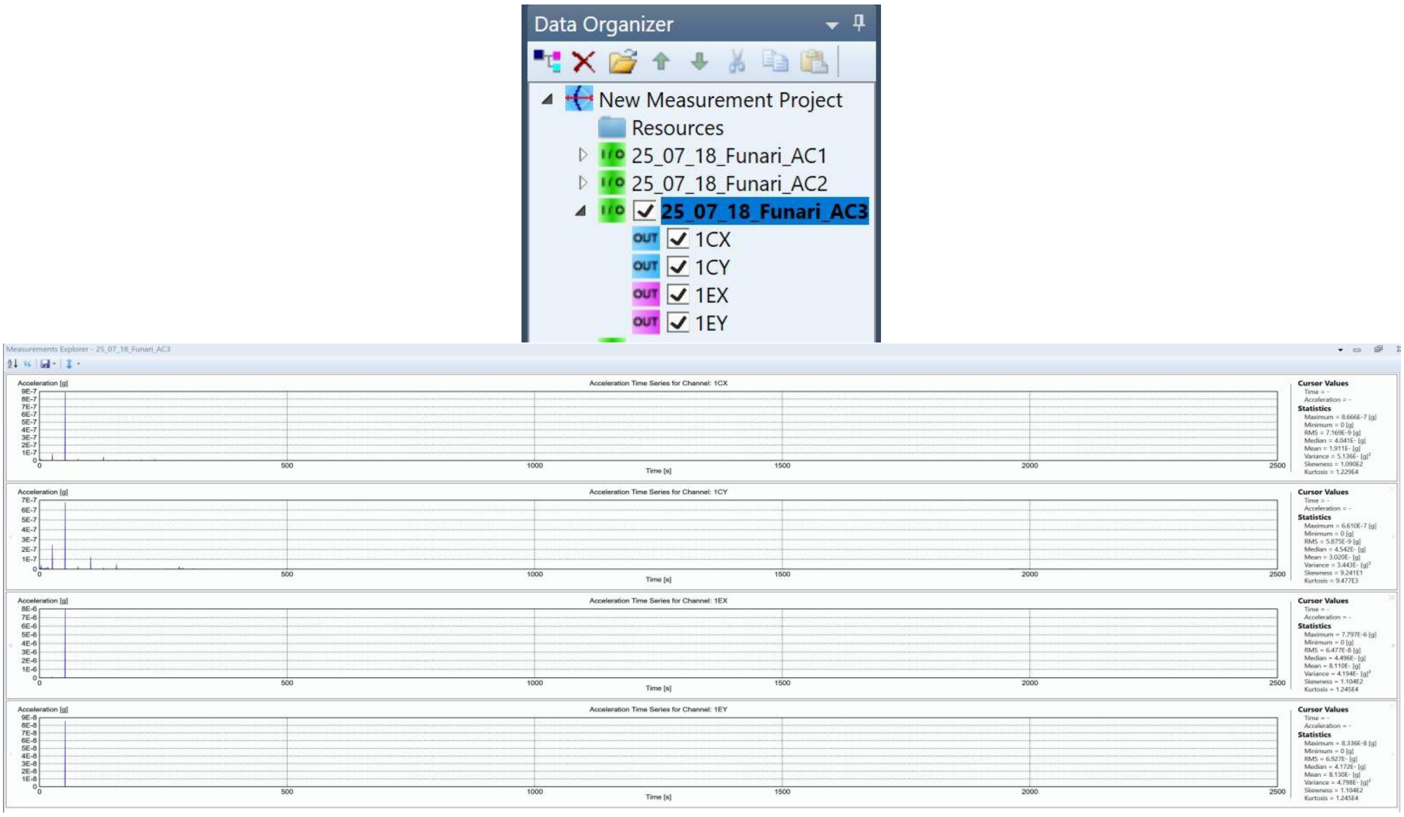


Figure 57. Test setup item 3

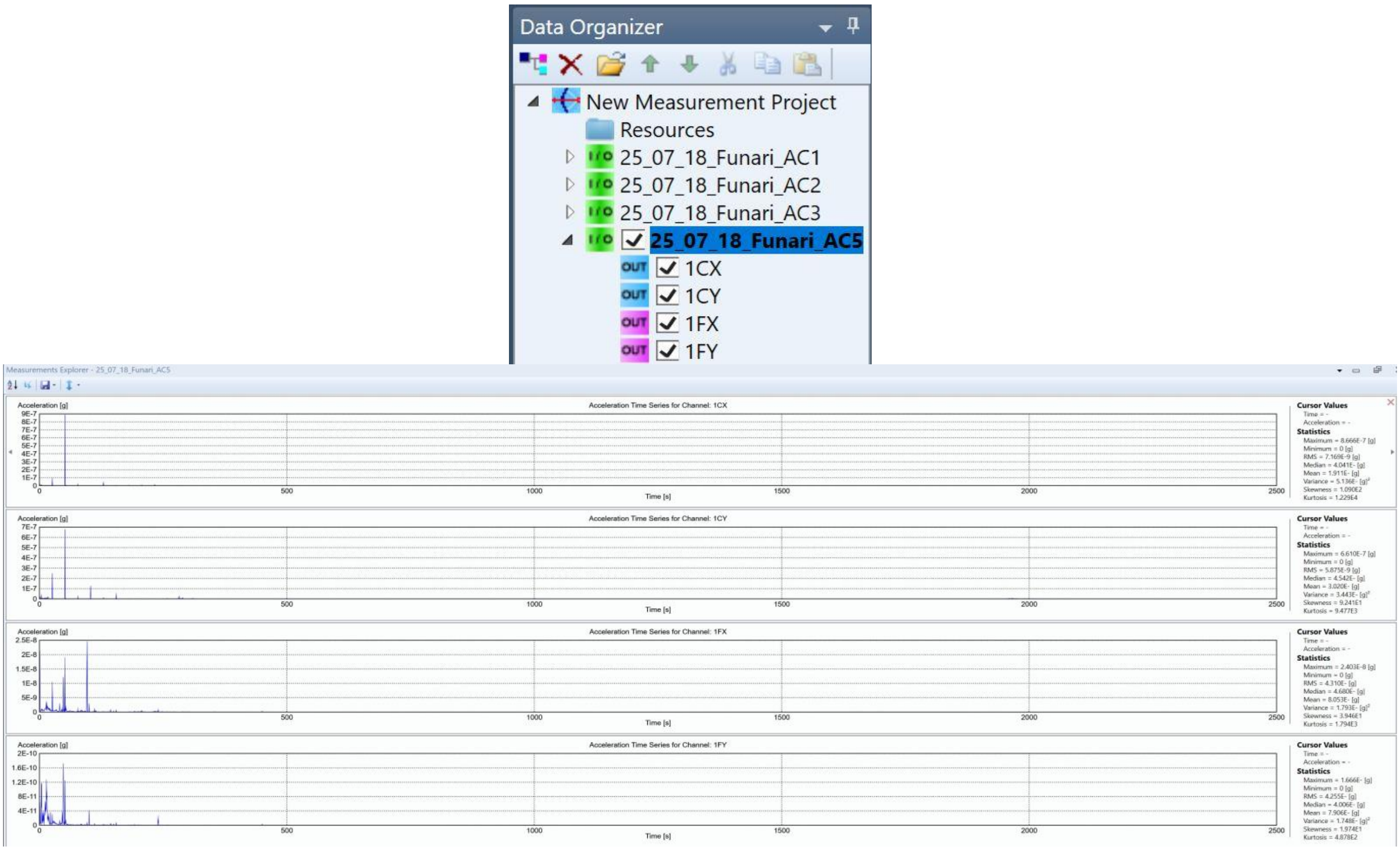


Figure 58. Test setup item 4

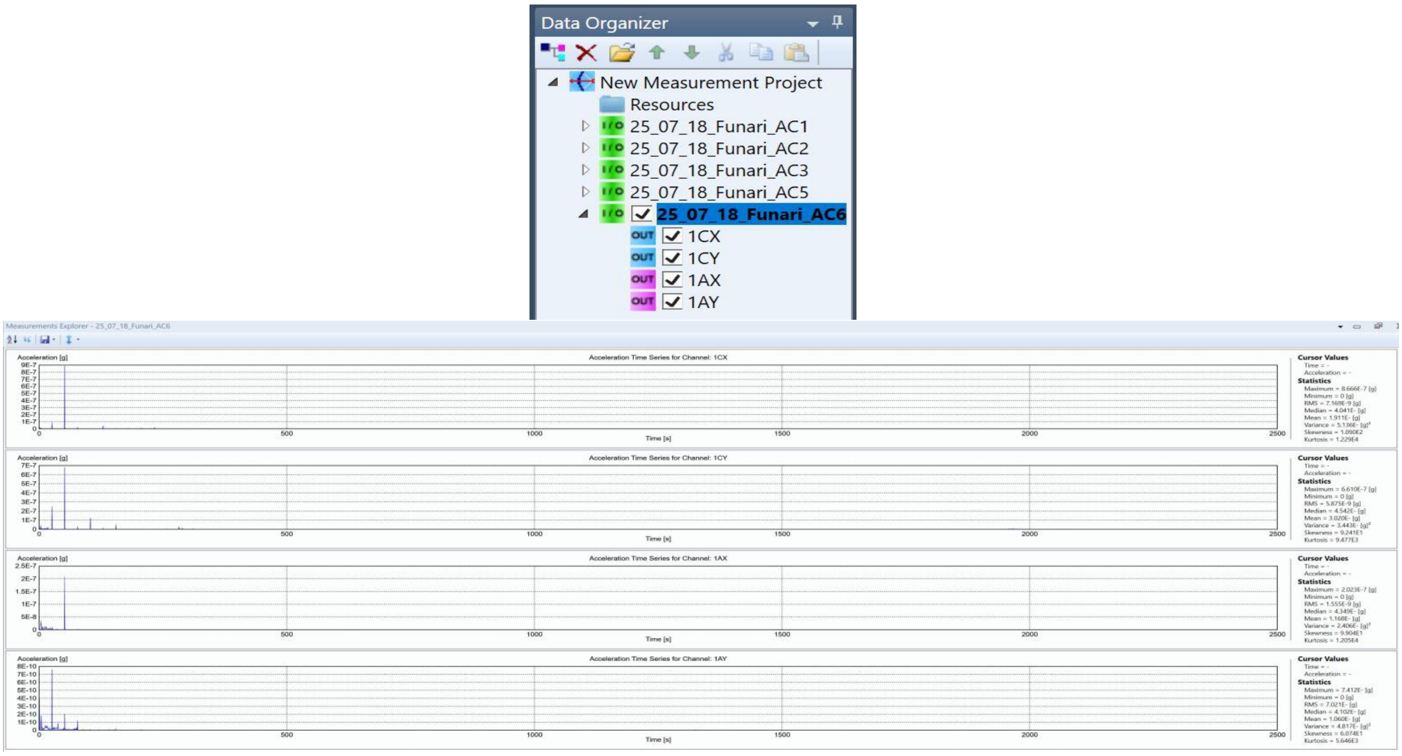
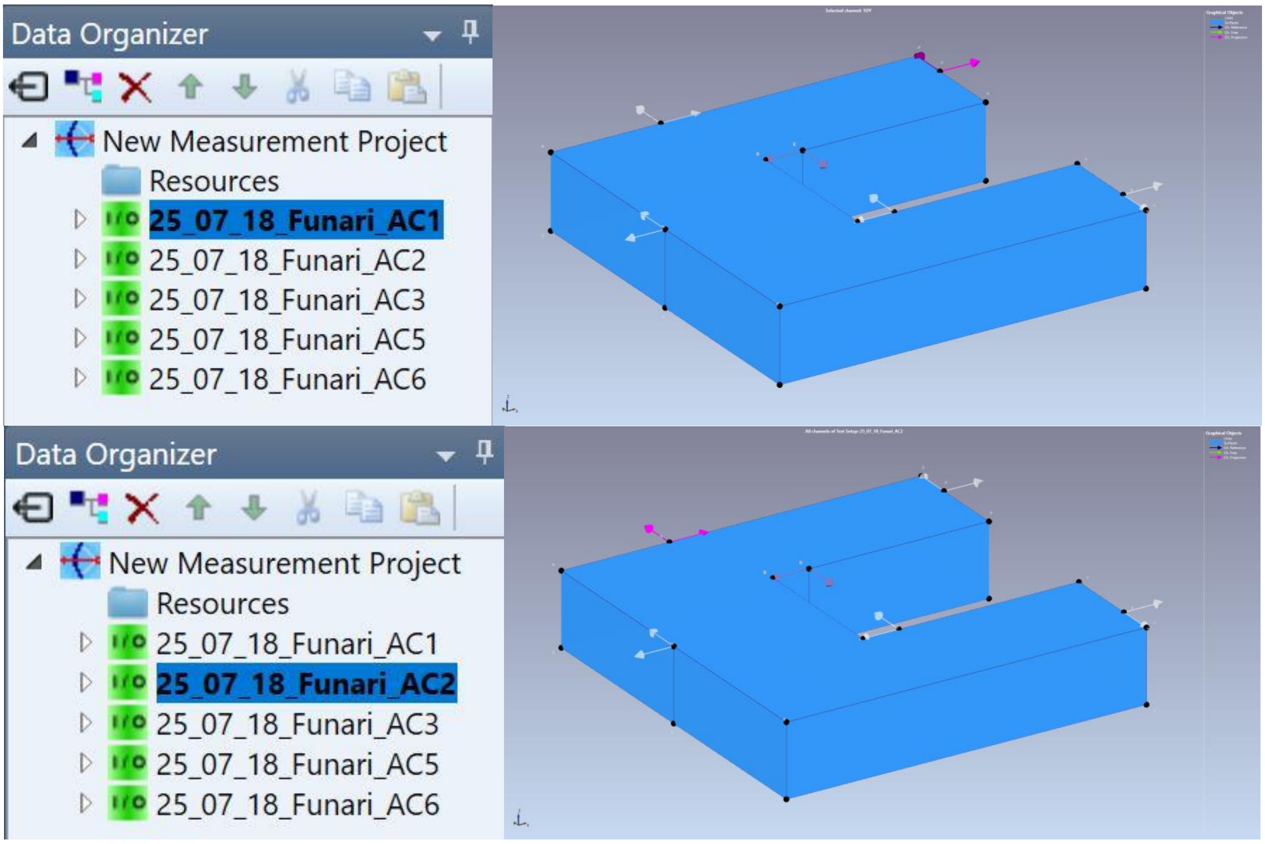


Figure 59. Test setup item 5

A test setup is defined as a specific configuration of measurement channels at specific locations and with specific directions.



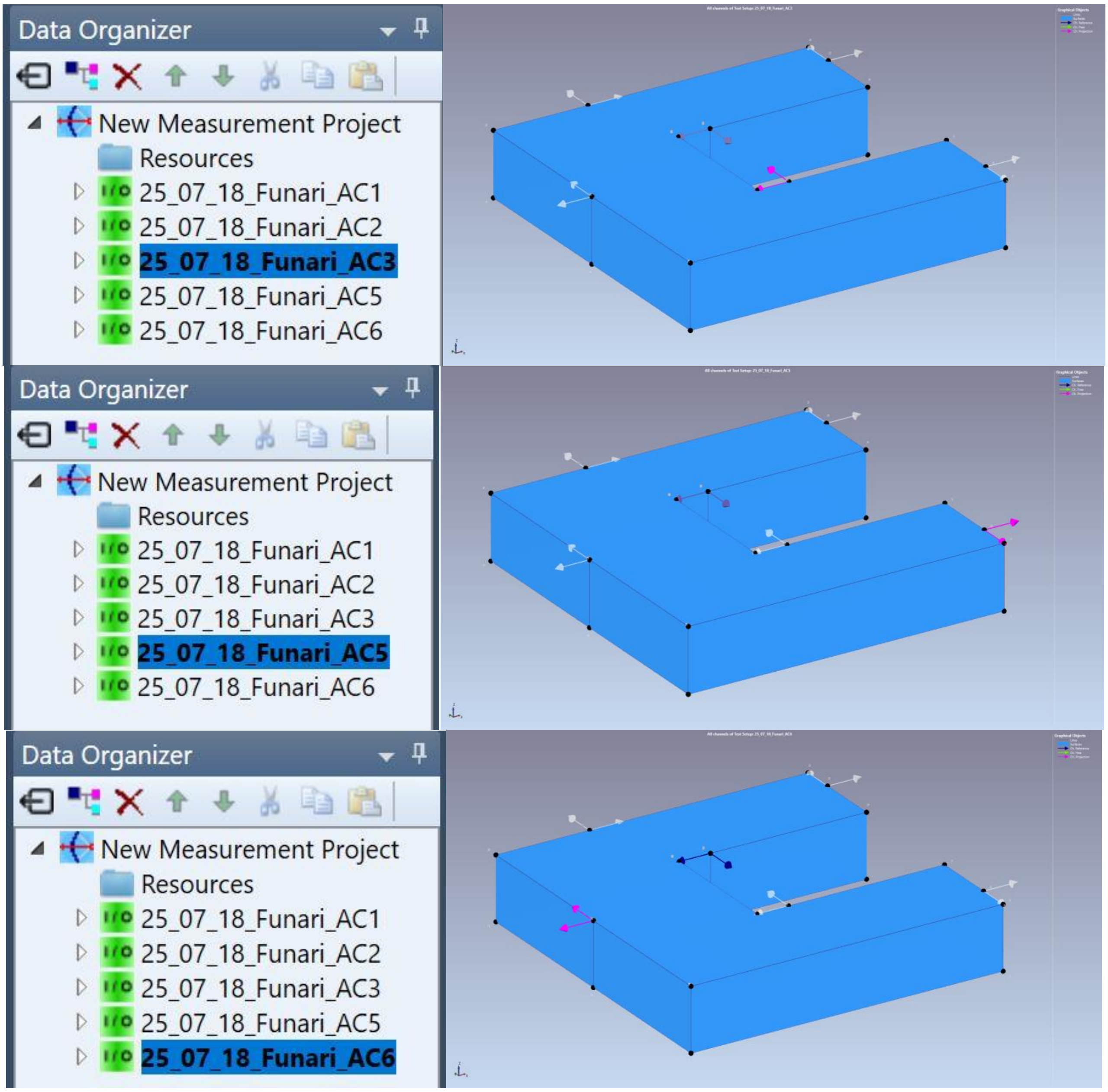


Figure 60. Assignment of DOF information

In the assignment of the DOF information task the idea is to mount the uploaded channel on the geometry in the right nodal points and in the right direction.

Once geometry and measurements are put together, I am ready to start signal processing the measurements in the various ways needed in order to perform modal analysis.

Before proceeding with the processing of the acquired data, some preliminary checks and treatments must be performed.

In particular, any anomalous trend must be controlled and any linear trend of the signal must be eliminated, given that it has no physical meaning because accelerometers are mounted on structures characterized by a net zero acceleration.

Multiple Test Setups are present, therefore the singular values calculated for each test setup were averaged to obtain the under displayed curves.

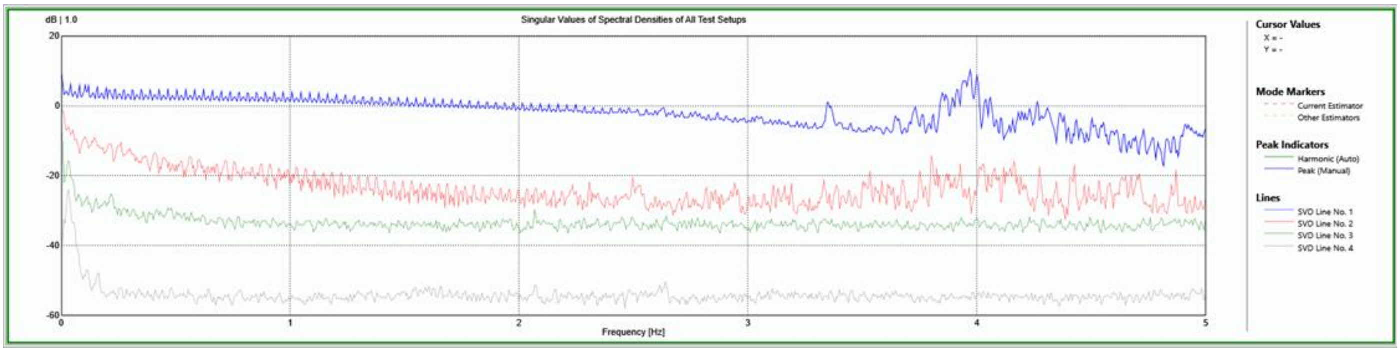


Figure 61. Singular values of spectral densities of all test setups

There are different analysis methods to use.

The choice of the most appropriate analysis method depending on the advantages and limitations related to data processing procedures.

The idea of using the Frequency Domain Decomposition (FDD) technique is that of performing an approximate decomposition of the system response into a set of independent single degree of freedom (SDOF) systems, one for each mode.

The theory is described in R. Brincker, L. Zhang and P. Andersen: Modal Identification from Ambient Responses using Frequency Domain Decomposition. Proc. of the 18th International Modal Analysis Conference (IMAC), San Antonio, Texas, 2000.

The decomposition is performed simply by decomposing each of the estimated spectral density matrices.

In the above reference it is shown that the singular values are estimates of the auto spectral density of the SDOF systems in modal coordinates, and in the vicinity of the resonance peak the singular vectors are estimates of the mode shapes of the mode.

The FDD technique involves the main steps listed below.

1. Estimate spectral density matrices from the raw time series data.

2. Perform singular value decomposition of the spectral density matrices.

3. If multiple test setups are available, then average the first singular value of all test setups and average the second etc.

4. Peak pick of the average singular values.

For well-separated modes always pick the first singular value.

In case of close or repeated modes, pick the second singular value, the third singular value etc. as well.

The technique is completely non-parametric and the modes are estimated purely by signal processing.

After completion of data signal processing, I can estimate the peaks corresponding to the first three ways and the results of the identification in terms of natural frequencies.

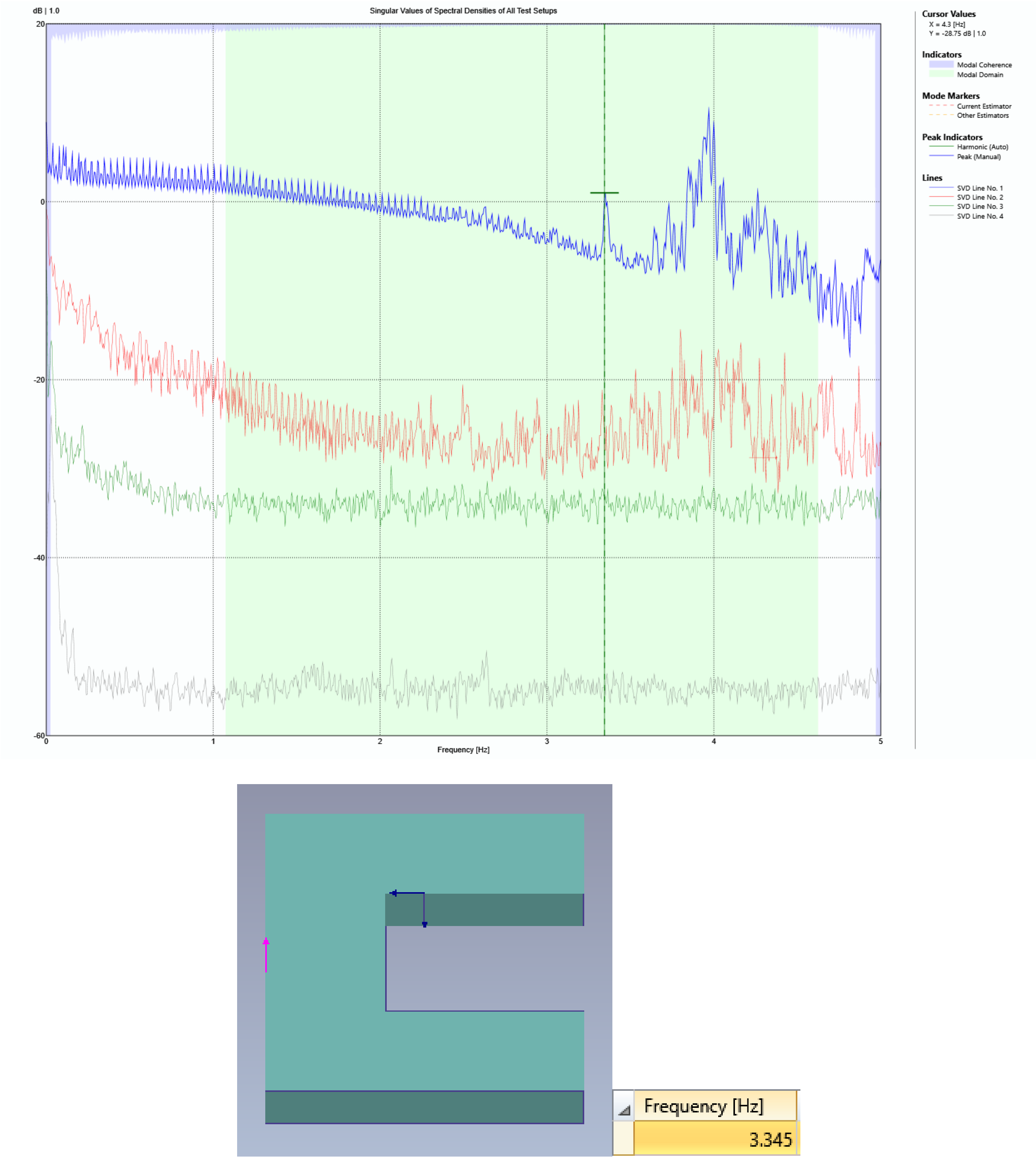
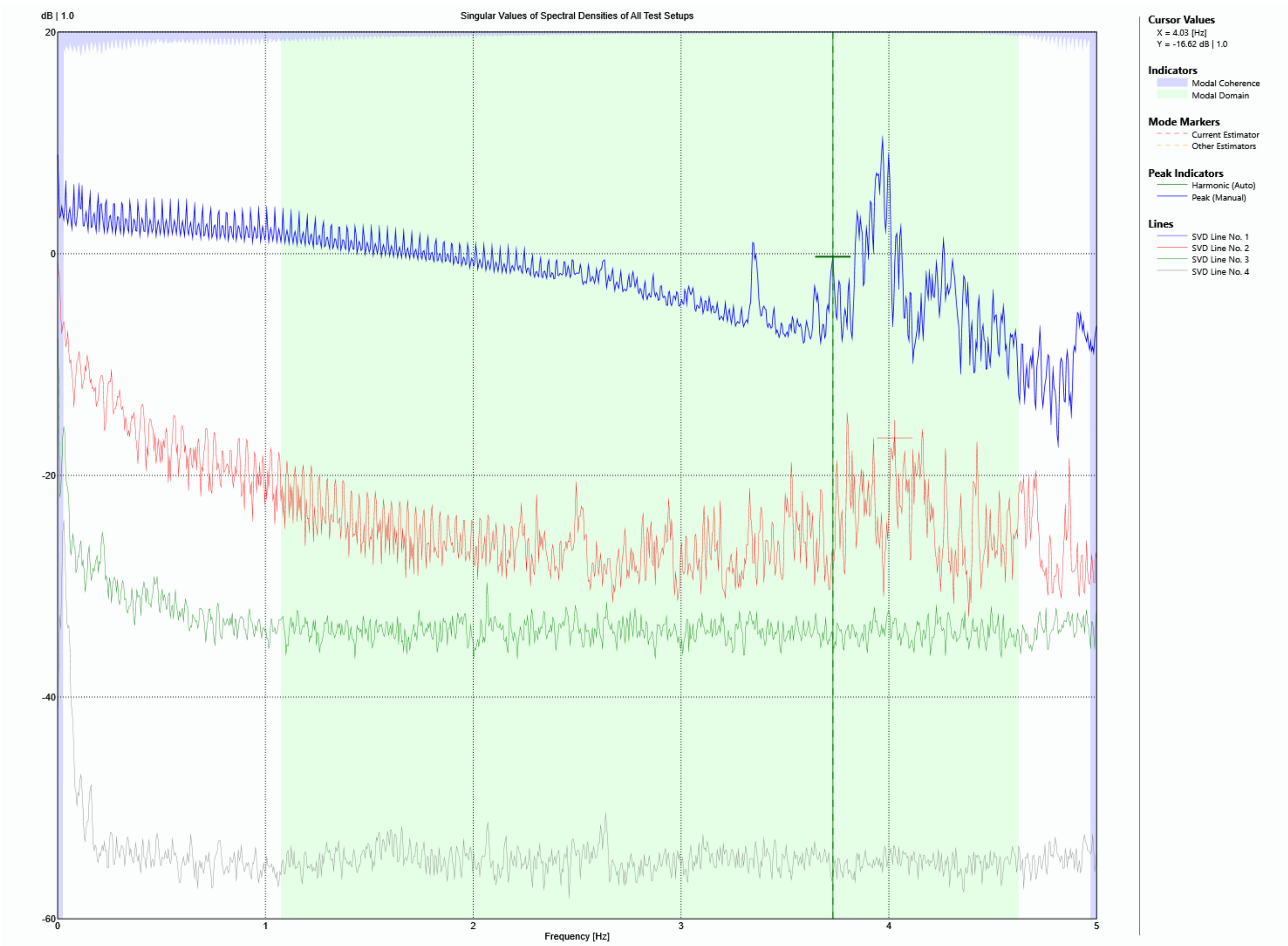


Figure 62. Mode 1 – T=0.30_Transversal



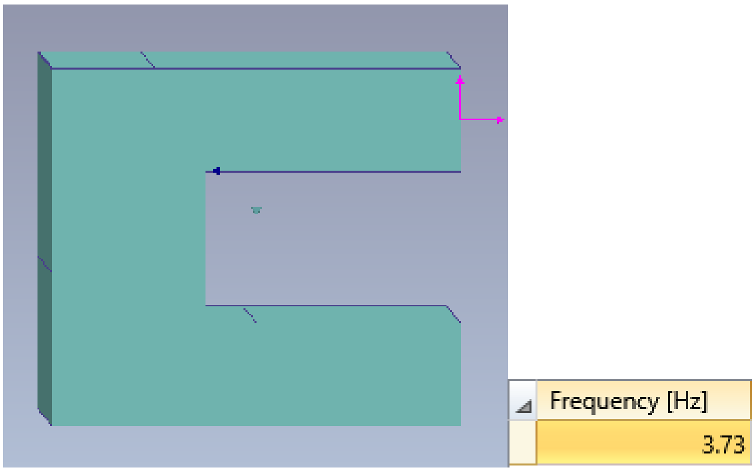


Figure 63. Mode 2 – T=0.27_Longitudinal and transversal

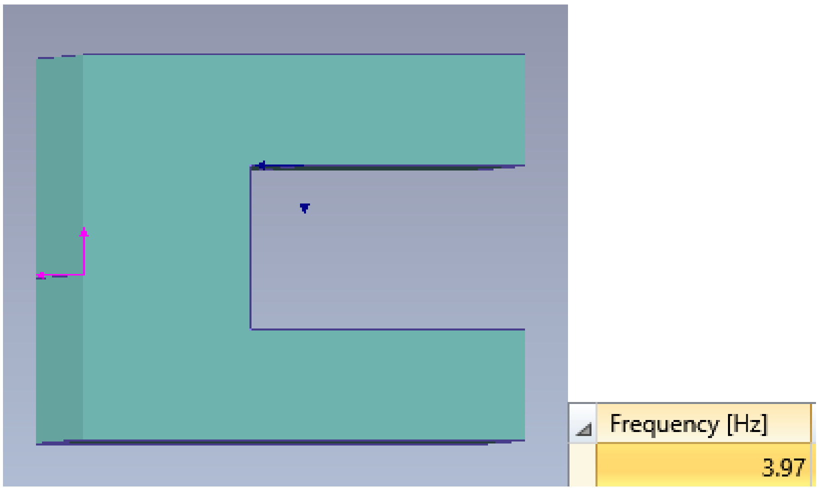


Figure 64. Mode 3 – T=0.25_Longitudinal

As can be seen from the modal shapes and natural frequencies, the first mode of vibration is transversal, the second mode of vibration is longitudinal and transversal, while the third mode is longitudinal.

4.7. Comparison

Figure 65-a illustrates the comparison between real experimental values and fem model values (theoretical) obtained <u>by modeling horizontal structures as **deformable planes**</u>.

The comparison concerned the values of natural frequencies and the percentage deviations of the obtained frequencies :

MODE	EXPERIMENTAL	FEM	Δf
	frequency (Hz)	frequency (Hz)	(%)
1	3,34	1,30	**61,08**
2	3,73	2,74	**26,54**
3	3,97	3,01	**24,18**

Figure 65-a. Comparison of results (OMA vs. DEFORMABLE PLANES MODEL)

This is the comparison of modal forms of the first three vibration modes:

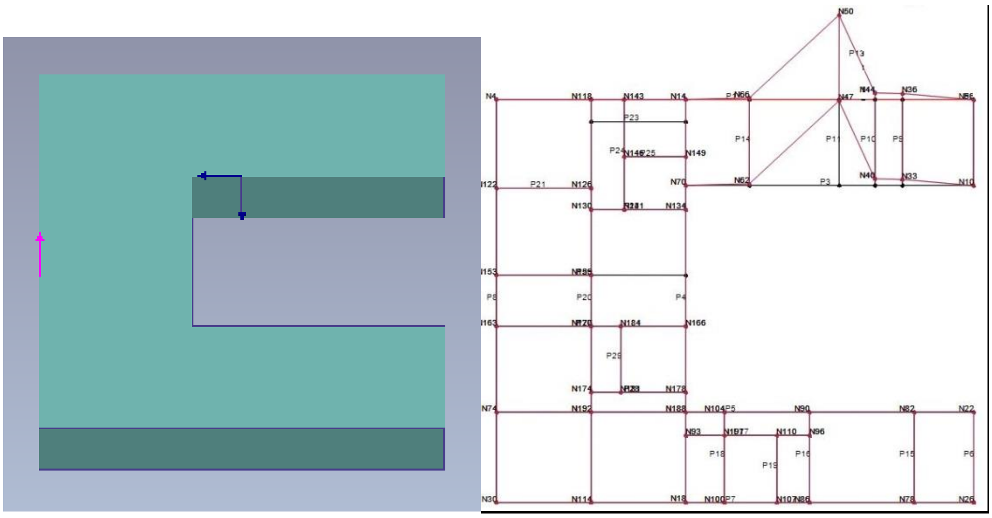


Figure 65-b. Mode 1 (OMA vs. DEFORMABLE PLANES MODEL)

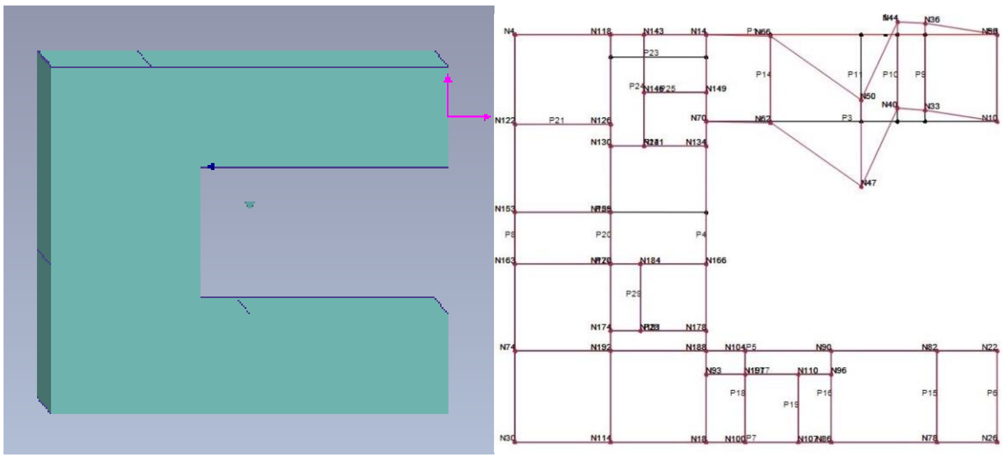


Figure 65-c. Mode 2 (OMA vs. DEFORMABLE PLANES MODEL)

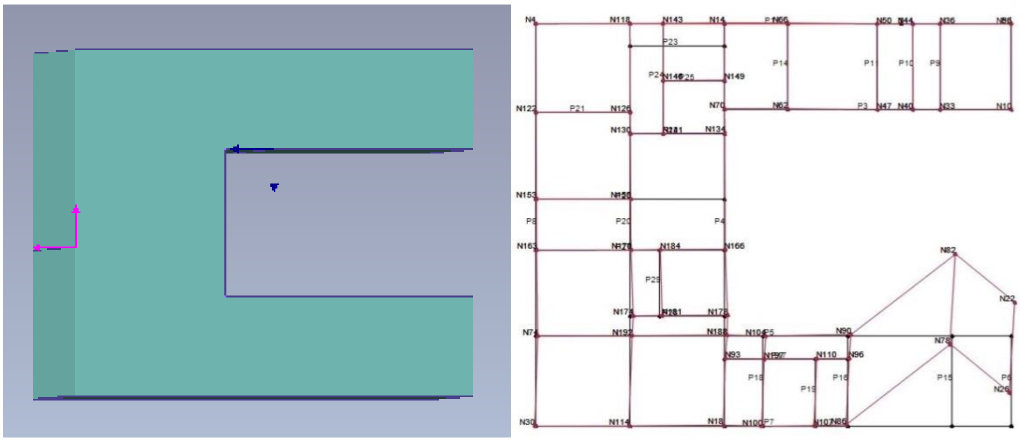


Figure 65-d. Mode 3 (OMA vs. DEFORMABLE PLANES MODEL)

Specifically, from the comparison between real values and theoretical values, the main differences are:

- the Δf(%) of all modes;
- the type of all modal forms.

The use of deformable bodies determined an irregular behavior due to a worse distribution of the actions through the horizontal diaphragms and a worse global response of the building, for which the adoption of low stiffness is inadequate.

Figure 66-a illustrates the comparison between real experimental values and fem model values (theoretical) obtained <u>by modeling horizontal structures as **rigid planes**</u>.

The comparison concerned the values of natural frequencies and the type of modal forms of the first three vibration modes; the percentage deviations of the obtained frequencies are also reported:

MODE	EXPERIMENTAL frequency (Hz)	FEM frequency (Hz)	Δf (%)	TYPE (EXPERIMENTAL)	TYPE (FEM)
1	3,34	3,54	-5,99	transversal	transversal
2	3,73	4,02	-7,77	**longitudinal and transversal**	**torsional**
3	3,97	4,19	-5,54	longitudinal	longitudinal

Figure 66-a. Comparison of results (OMA vs. RIGID PLANES MODEL)

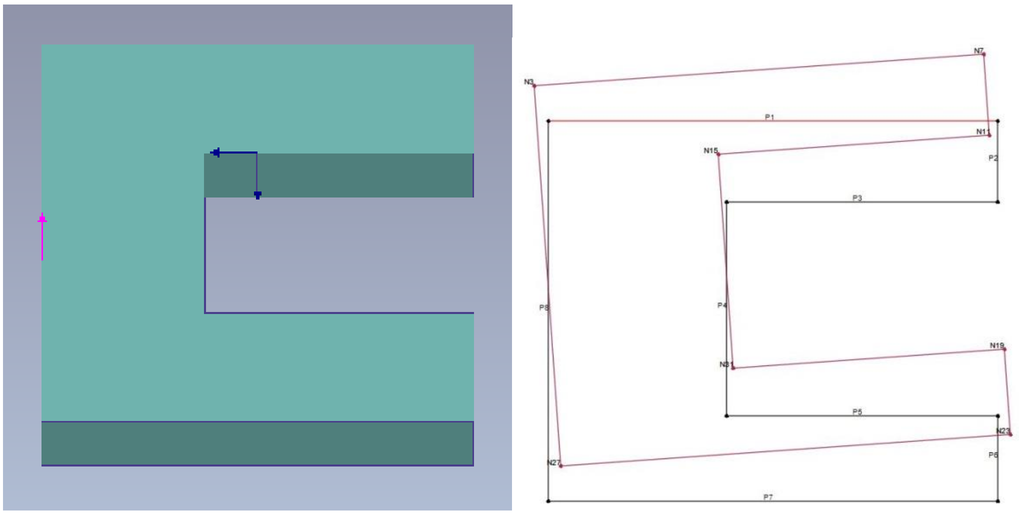


Figure 66-b. Mode 1 (OMA vs. RIGID PLANES MODEL)

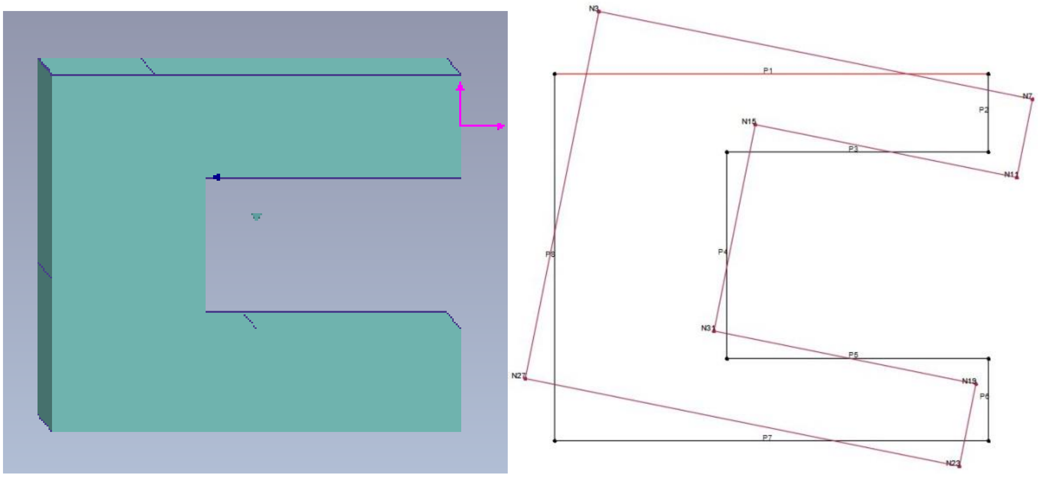


Figure 66-c. Mode 2 (OMA vs. RIGID PLANES MODEL)

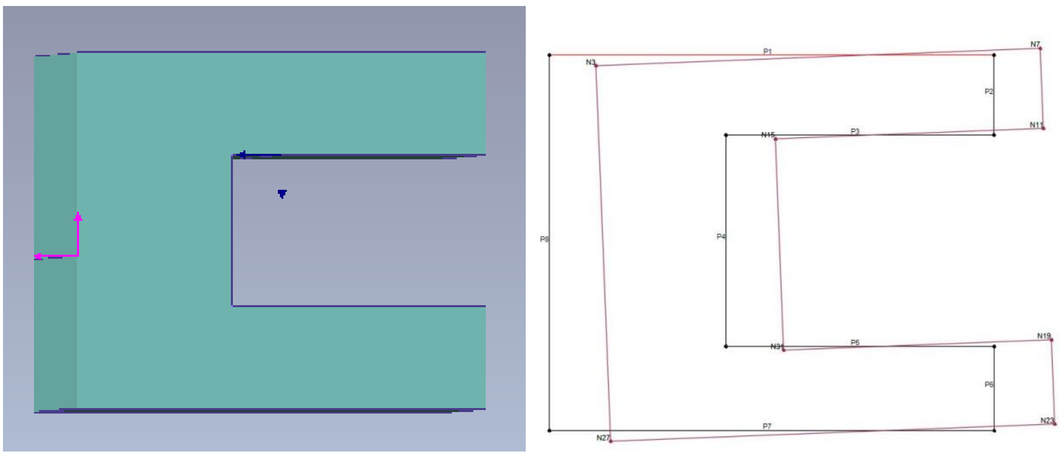


Figure 66-d. Mode 3 (OMA vs. RIGID PLANES MODEL)

Specifically, from the comparison between real values and theoretical values, the main difference is the type of the second modal form, while the other values are almost concordant.

As shown by the tables below, the difference between deformable or rigid diaphragms is significant. Indeed, the periods of vibration and the participating masses change significantly.

Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0,77009	0	0,00	109.575	3,84	0	0,00
2	0,36528	17	0,00	16.247	0,57	4	0,00
3	0,33256	2.570	0,09	228.397	8,00	38	0,00

Figure 35*. List of modal forms – *DEFORMABLE PLANES*

Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0,28264	19.332	0,68	1.899.657	66,50	35	0,00
2	0,24878	150.071	5,25	94.619	3,31	11	0,00
3	0,23882	2.068.052	72,40	61.326	2,15	41	0,00

Figure 39*. List of modal forms – *RIGID PLANES*

This modeling aspect is particularly relevant for those assets characterized also by the partial presence of wooden and vaulted floors, like the examined one under examination, for which the adoption of infinite stiffness is, however, inadequate.

For this reason, the adoption of semi-rigid diaphragms obtained by using an intermediate stiffness, was found to be the most appropriate and realistic one for subsequent application.

Figure 67-a illustrates the comparison between real experimental values and fem model values (theoretical) obtained by modeling horizontal structures as **semi-rigid planes**.

Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0,45254	385	0,02	1.508.933	81,59	213	0,01
2	0,29229	43.584	2,36	206.320	11,16	115	0,01
3	0,24880	1.607.302	86,91	4.018	0,22	20	0,00

Figure 67-a. List of modal forms – SEMI-*RIGID PLANES*

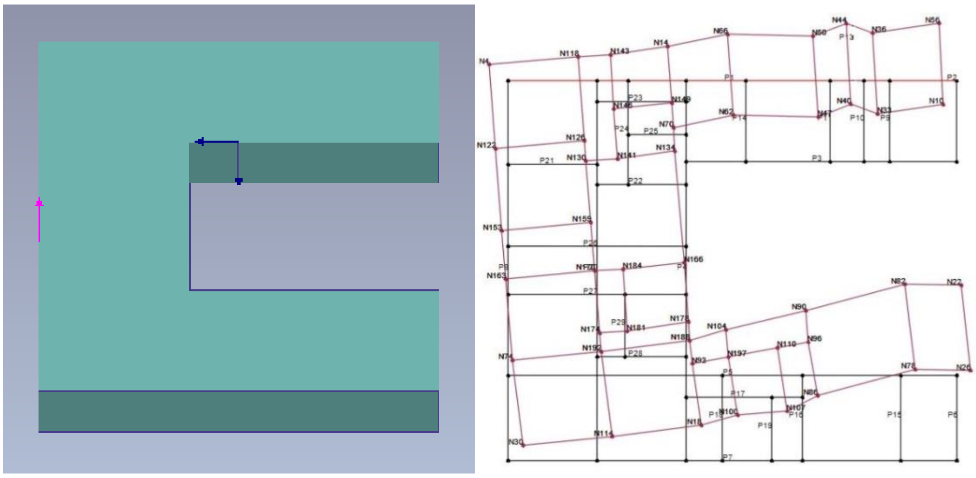


Figure 67-b. Mode 1 (OMA vs. SEMI-RIGID PLANES MODEL)

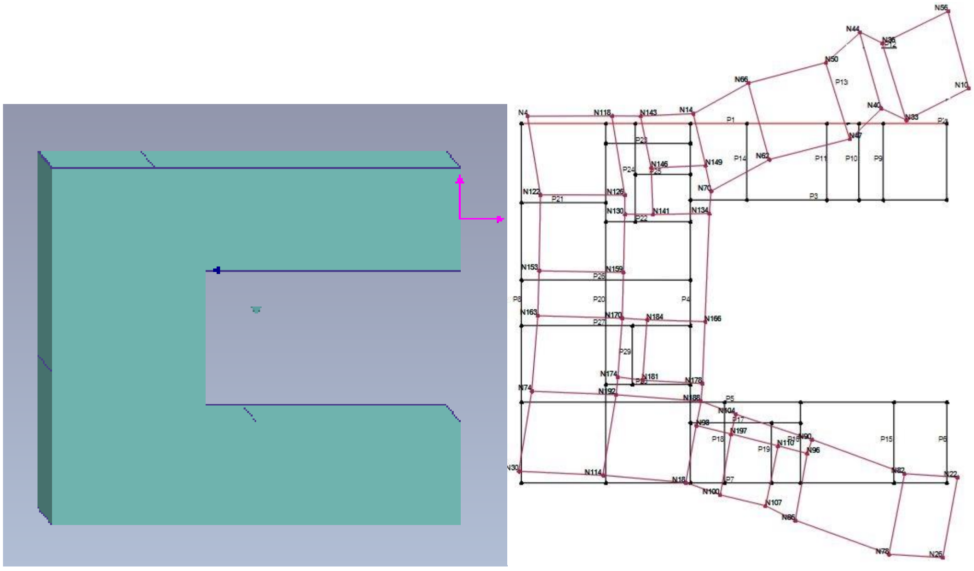


Figure 67-c. Mode 2 (OMA vs. SEMI-RIGID PLANES MODEL)

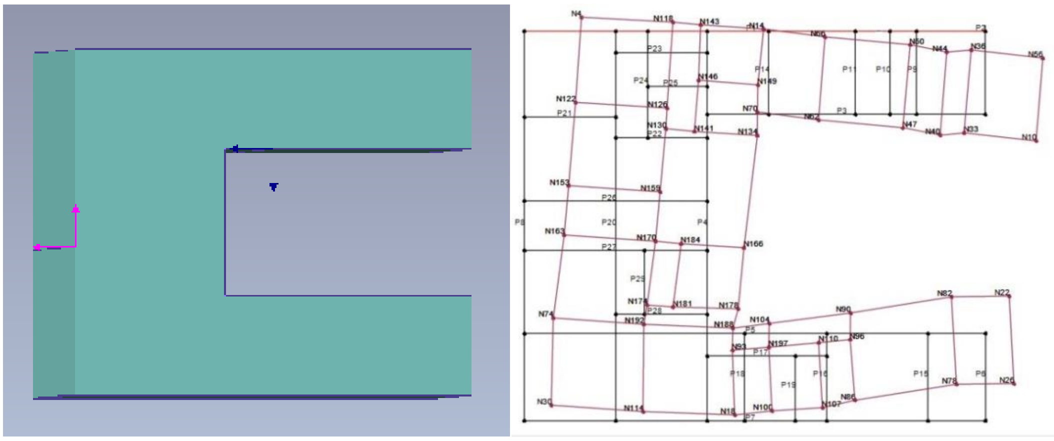


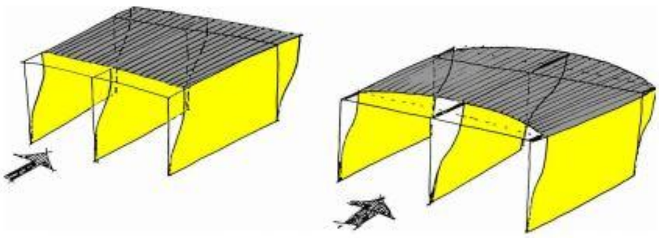
Figure 67-d. Mode 3 (OMA vs. SEMI-RIGID PLANES MODEL)

MODE	EXPERIMENTAL frequency (Hz)	FEM frequency (Hz)	Δf (%)
1	3,34	2,21	33,83
2	3,73	3,42	8,31
3	3,97	4,02	-1,26

Figure 67-e. Comparison of results (OMA vs. SEMI-RIGID PLANES MODEL)

Thanks to the results obtained, I was able to calibrate the parameters initially used, improving the preliminary knowledge.

The real experimental values obtained allowed me to understand that in preliminary modelling I had not calibrated the stiffness correctly, considering the diaphragms to be too deformable or too rigid.



In conclusion, it can be affirmed that the imperfect compatibility of the results can be due to:

• modelling errors due to uncertain knowledge of the current state (history, materials, details);

• structure that differs from the project due to degradation and damage.

These circumstances, which can significantly reduce the strength and stiffness of the structural elements, together with residual uncertainties about the constructive details such as the effectiveness of good connections between the walls, were studied through a sensitivity analysis.

4.8. Sensitivity analysis

Sensitivity analysis is a calculation method aimed at achieving a better understanding of structural functioning and an accurate planning of the site investigation plan.

As is known, doubts during modelling directly affect the evaluation of seismic safety.

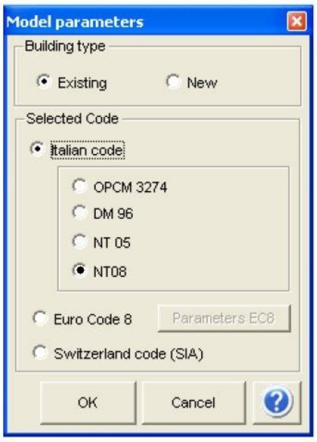
A specific example is the mechanical properties of the materials, generally defined on the basis of reference values.

Investigations aim to limit the inevitable uncertainty.

The sensitivity analysis methodology includes the identification of groups of parameters expressing the degree of uncertainty.

The execution of several different non-linear analyses identifies a level of sensitivity for each parameter in order to provide a weight in terms of importance.

Tremuri performs static non-linear analyses on masonry buildings using the current Italian building code: Decree 14/01/2008 [6] (recently updated by NTC 2018). [29]



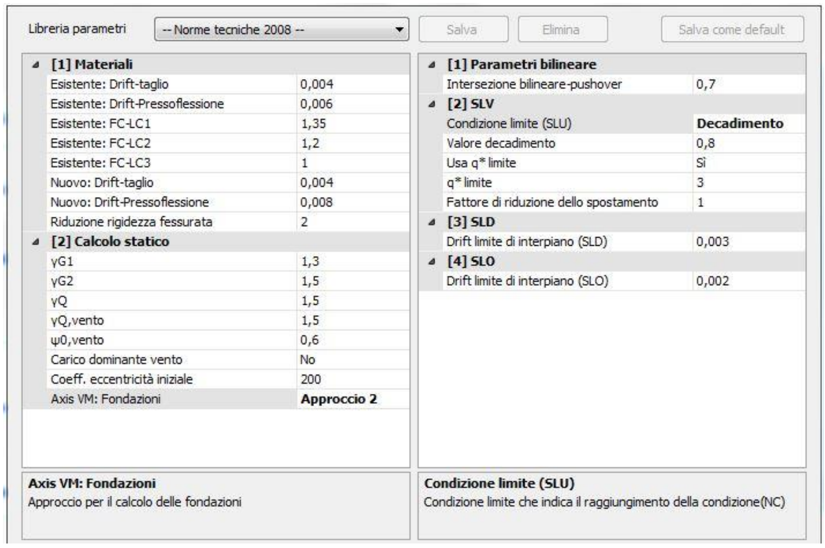


Figure 68. Reference code

The process to be followed in the verification of the structure to be examined consists of the following steps:

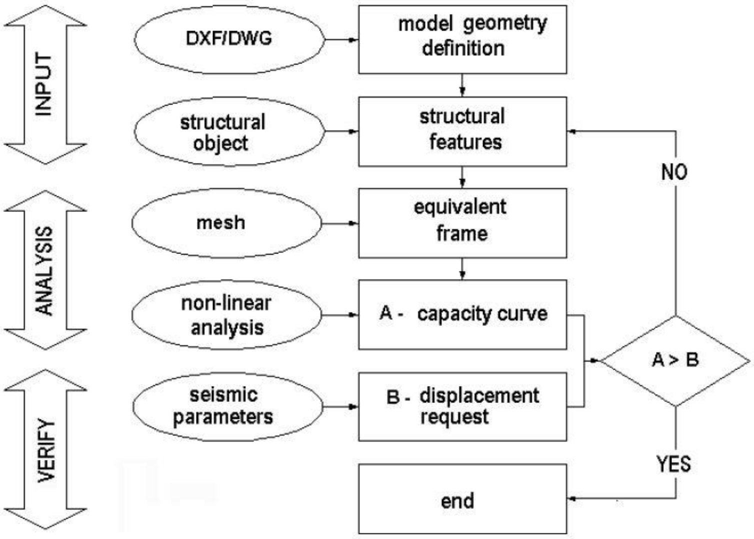


Figure 69. The procedure of Tremuri software

The input of the analysis groups and parameters to be checked is required.

First, an equivalent frame model is automatically created, non-linear static analyses (push-over) follows, from which the structural capacity curves are derived (strain curve - displacement of the control point).

The use of non-linear static analysis (pushover) is necessary to characterize the seismic-resistant system through capacity curves: "static" analysis where the external force is applied to the structure statically, and "non-linear" due to the behavioral model used for the structural resistance elements.

These curves are intended to represent the envelope of the hysteresis cycles produced during the seismic event and can be considered to be an indicator of the post-elastic behavior of the structure.

In this way, in the elastic analysis methods, the non-linear behavior is taken into account by introducing the structural factor.

Non-linear static analysis does not allow the structural response to evolve as each single element evolves in the non-linear field, providing information on the distribution of the inelasticity demand.

The curve obtained by the pushover analysis (which will then by transformed into a capacity curve, taking into account the system characteristics equivalent to degrees of freedom) conventionally provides information on the trend of the shear resulting at the base, with respect to the horizontal displacement of a control point on the structure.

At each point on the curve, a specific damage state for the entire system can be linked, and so it is possible to link determined displacement levels to the level of expected performance and the corresponding damage.

The curve is obtained by using pushover analysis, which predicts the assignment of a preset distribution of forces increasing in a static and monotonic manner.

The distribution is kept unaltered even after the fail limit is reached.

The analysis can also be conducted by controlling for forces or for mixed force-displacement.

The load distribution applied is intended to represent the distribution of inertial forces induced by the seismic event.

The profiles proposed are those in harmony with the first modal form, for masonry structures, more or less equivalent to those adopted for the linear static analysis, and proportional to the mass. In particular, in the case of regular structures, the first distribution is chosen with the intention of better determining the structural response in the elastic field and secondly, in the non-linear field.

The "capacity" offered by the structure must then be determined, through the lens of a seismic check, with the "demand" requested by the external force, that is by a determined seismic event.

The energy dissipation effects, which offer an ulterior margin of resistance, cannot be explained by only using linear elastic theory.

They are relevant in particular in the field of non-linear structural response: the demand is reduced by taking them into account.

The expected response for the building, as a function of a determined action, is hence obtained through the identification of the performance point (whose coordinates in terms of spectrum displacement corresponds to d*max).

The maximum displacement value that can be offered by the building in a seismic event, is obtained in correspondence with the value of the shear that underwent a decline of 20% from the shear limit value.

Based on the capacity curve of the real system defined in this way, it passes to the bilateral associated with the equivalent system; once found, the system period with one degree of freedom is identified, whose behavior permits the identification of the seismic event's displacement demand.

Definition of seismic parameters and evaluation of the parameters derived from the structure's capacity curve permits determination of the request in terms of displacement of the spectrum for the project at hand.

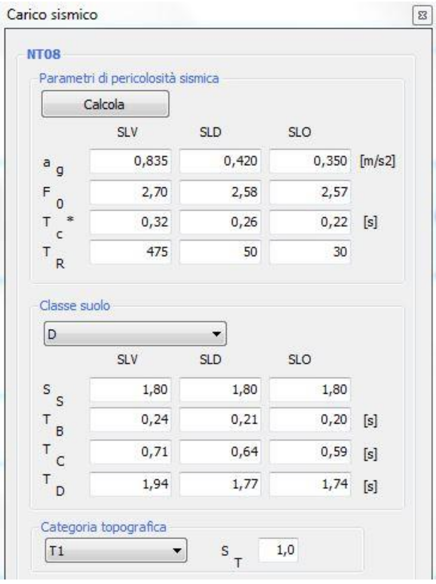


Figure 70. Seismic parameters definition

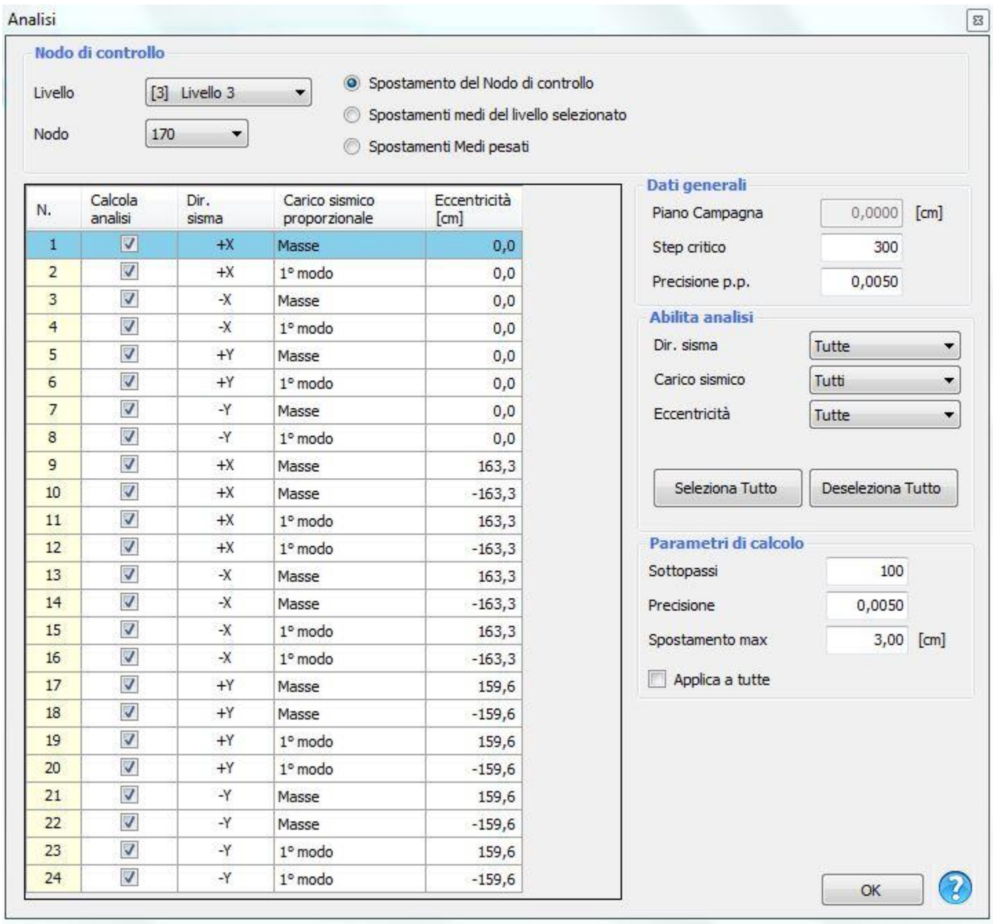


Figure 71. Calculation parameters definition

Verifica analisi

N.	Inserisci in relazione	Dir. sisma	Carico sismico proporzionale	Eccentricità [cm]	Dmax SLV [cm]	Du SLV [cm]	q* SLV	Dmax SLD [cm]	Du SLD [cm]	Dmax SLO [cm]	Do SLO [cm]	α SLV	α SLD	α SLO
1	☑	+X	Masse	0,00	1,89	0,74	2,38	0,52	0,52	0,35	0,52	0,562	0,999	1,218
2	☑	+X	1° modo	0,00	2,26	0,49	3,40	0,76	0,31	0,53	0,31	0,365	0,632	0,764
3	☑	-X	Masse	0,00	1,68	2,43	1,66	0,46	2,34	0,38	2,34	1,270	2,719	3,425
4	☑	-X	1° modo	0,00	2,17	0,60	2,49	0,63	0,52	0,42	0,52	0,463	0,912	1,107
5	☑	+Y	Masse	0,00	0,40	0,42	1,20	0,16	0,35	0,13	0,35	1,028	1,972	2,399
6	☑	+Y	1° modo	0,00	0,44	0,45	1,20	0,18	0,45	0,15	0,45	1,007	2,135	2,613
7	☑	-Y	Masse	0,00	0,33	0,57	0,93	0,16	0,57	0,13	0,57	1,344	2,858	3,507
8	☑	-Y	1° modo	0,00	0,41	0,76	1,06	0,18	0,58	0,15	0,44	1,422	2,569	2,650
9	☑	+X	Masse	163,25	1,67	1,10	1,83	0,42	1,08	0,35	1,08	0,783	1,642	2,031
10	☑	+X	Masse	-163,25	1,81	0,52	2,11	0,45	0,52	0,36	0,52	0,512	1,061	1,287
11	☑	+X	1° modo	163,25	2,22	1,04	3,16	0,72	0,95	0,50	0,95	0,578	1,192	1,490
12	☑	+X	1° modo	-163,25	2,26	0,70	3,21	0,74	0,63	0,51	0,63	0,451	0,907	1,119
13	☑	-X	Masse	163,25	1,70	1,62	1,70	0,46	0,86	0,38	0,86	0,974	1,480	1,809
14	☑	-X	Masse	-163,25	1,77	2,07	1,80	0,46	2,07	0,38	2,07	1,107	2,429	3,056
15	☑	-X	1° modo	163,25	2,12	0,62	2,38	0,60	0,53	0,42	0,53	0,484	0,945	1,145
16	☑	-X	1° modo	-163,25	2,10	0,51	2,28	0,57	0,51	0,42	0,51	0,455	0,950	1,148
17	☑	+Y	Masse	159,59	0,42	0,27	1,39	0,14	0,24	0,12	0,24	0,800	1,588	1,922
18	☑	+Y	Masse	-159,59	0,37	0,99	1,02	0,17	0,60	0,14	0,60	1,776	2,756	3,393
19	☑	+Y	1° modo	159,59	0,48	0,27	1,53	0,16	0,24	0,13	0,24	0,723	1,431	1,730
20	☑	+Y	1° modo	-159,59	0,44	1,05	1,09	0,20	0,57	0,16	0,57	1,710	2,443	2,996
21	☑	-Y	Masse	159,59	0,49	0,90	1,79	0,14	0,20	0,12	0,20	1,563	1,271	1,543
22	☑	-Y	Masse	-159,59	0,37	1,05	0,93	0,18	0,99	0,15	0,75	1,877	3,919	4,005
23	☑	-Y	1° modo	159,59	0,44	0,51	1,27	0,16	0,34	0,14	0,34	1,090	1,841	2,238
24	☑	-Y	1° modo	-159,59	0,42	1,08	1,01	0,20	0,72	0,17	0,54	1,759	2,878	2,925

Legenda colori

Verificato — Non verificato — Non converge a p.p. — Analisi più gravosa

Figure 72. Seismic calculations results (preliminary model)

The check compares the two displacements offered by the structure and required by the code.

If the first is greater than the second, the structure satisfies the check.

This window shows the results of the seismic computations performed on the model and summarizes the check parameters according to each norm, indicating whether the results were satisfactory or not.

The first columns describe the type of analysis, the last shows the vulnerability indexes for each of the three limit states.

The background color, green or red, distinguishes between the exceeded analysis by those that are not.

The yellow color shows the two analyzes that have the lowest vulnerability indexes (more significant for the purposes of calculation).

The analyzes that have minimum "Alfa" values are more restrictive, so the results window shows the two analyzes with the minimum "Alfa SLV" (one for the X direction and one for the Y direction).

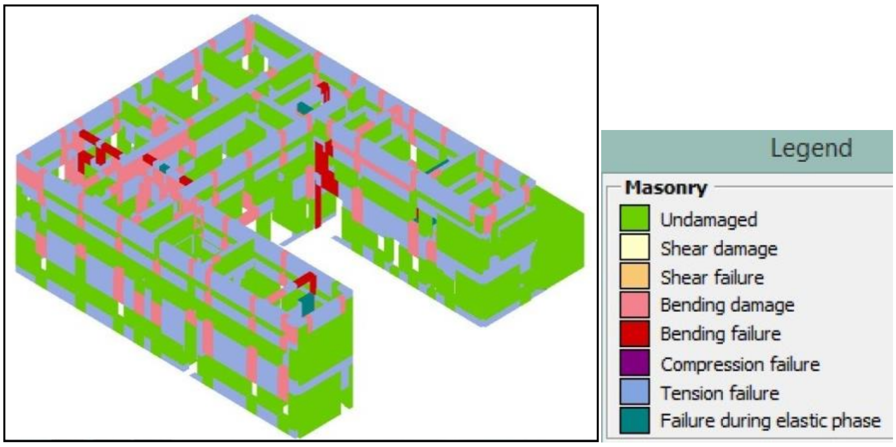


Figure 73. Analysis N.2 – “X” direction

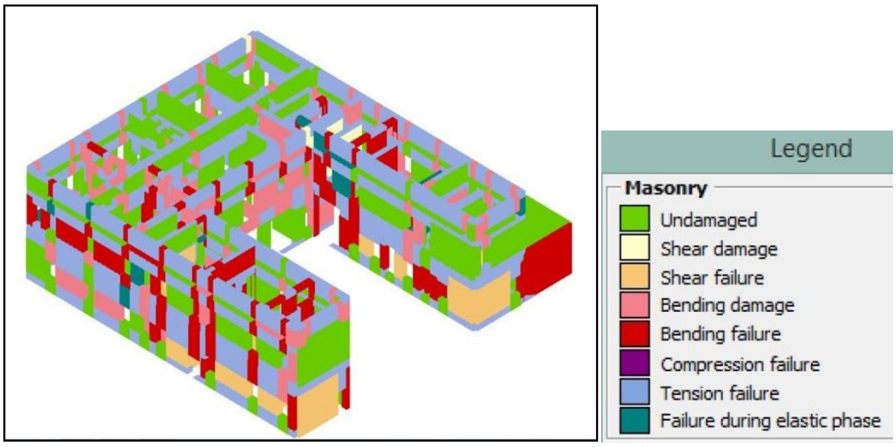


Figure 74. Analysis N.19 – “Y” direction

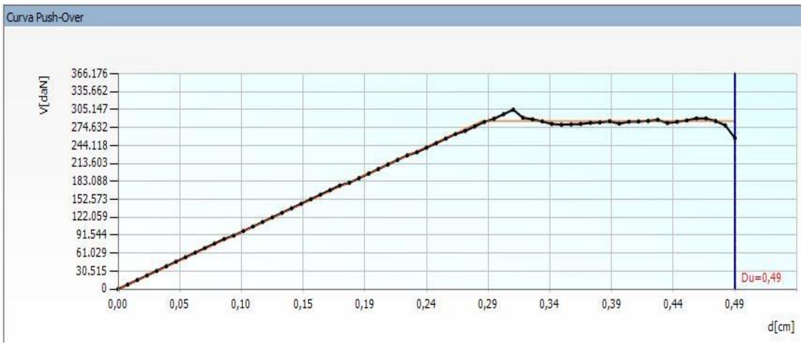


Figure 75. Analysis N.2 – “X” direction pushover curve

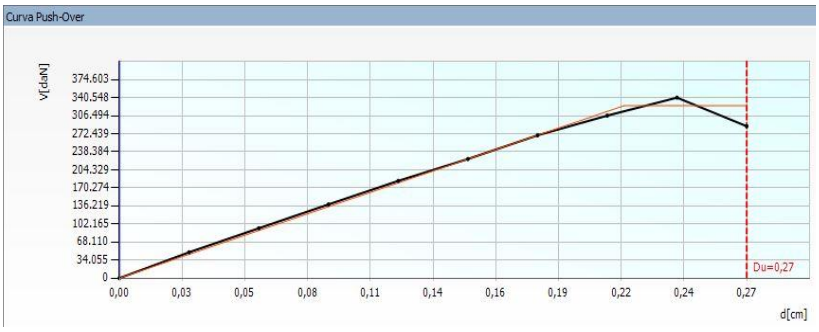


Figure 76. Analysis N.19 – “Y“ direction pushover curve

The pushover curve is shown in black, in orange the bilinear equivalent.

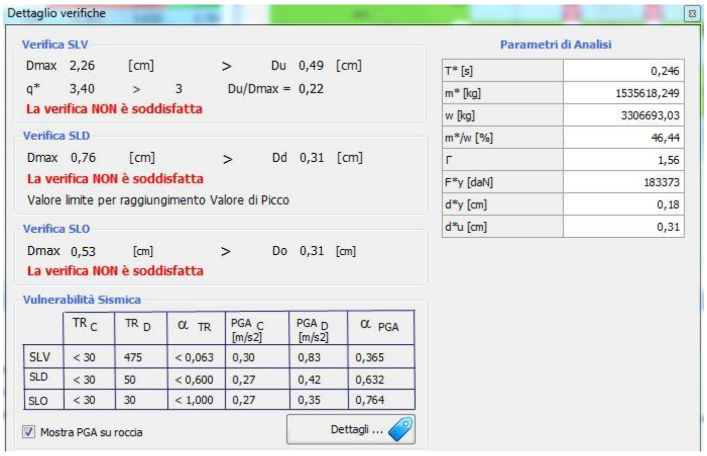


Figure 77. Analysis N.2 – “X” direction results details

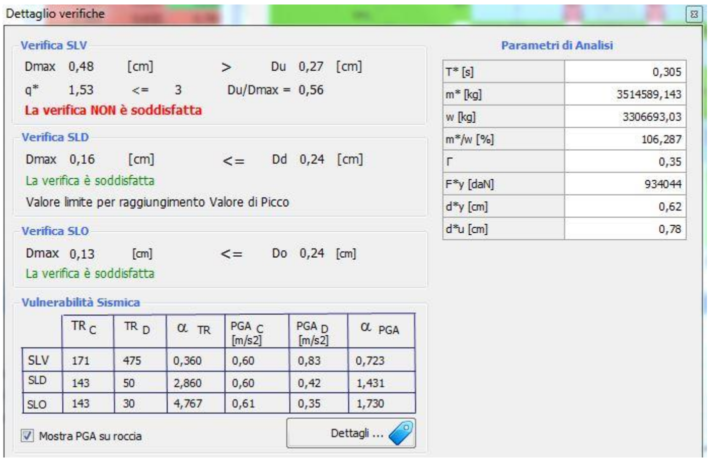


Figure 78. Analysis N.19 – “Y” direction results details

These represents two summary windows that display the details of the analyses and required checks.

Vulnerabilità Sismica

						TR_C			TR=cost	
	TR_C	TR_D	α_{TR}	PGA_D [m/s2]	$PGA_C(TR)$ [m/s2]	$F_0(TR)$	$T_C^*(TR)$	$\alpha_{PGA}(TR)$	PGA_C [m/s2]	α_{PGA}
SLV	< 30	475	< 0,063	0,83	0,00	0,00	0,00	0,000	0,30	0,365
SLD	< 30	50	< 0,600	0,42	0,00	0,00	0,00	0,000	0,27	0,632
SLO	< 30	30	< 1,000	0,35	0,00	0,00	0,00	0,000	0,27	0,764

Figure 79. Analysis N.2 – “X” direction seismic vulnerability

Vulnerabilità Sismica

						TR_C			TR=cost	
	TR_C	TR_D	α_{TR}	PGA_D [m/s2]	$PGA_C(TR)$ [m/s2]	$F_0(TR)$	$T_C^*(TR)$	$\alpha_{PGA}(TR)$	PGA_C [m/s2]	α_{PGA}
SLV	171	475	0,360	0,83	0,62	2,62	0,30	0,747	0,60	0,723
SLD	143	50	2,860	0,42	0,59	2,62	0,29	1,404	0,60	1,431
SLO	143	30	4,767	0,35	0,59	2,62	0,29	1,685	0,61	1,730

Figure 80. Analysis N.19 – “Y” direction seismic vulnerability

The tables for the "Seismic Vulnerability" evaluation show the α parameters derived from the homonyms reports for each of the limit states:

$\alpha_{PGA}=PGA_C/PGA_D$

$\alpha_{TR}=TR_C/TR_D$

- **PGA_C**: Limit capacity acceleration for each limit state (independent from the seismic spectrum).
- **PGA_D**: Spectral acceleration for each of the limit states (depends on the seismic spectrum).
- **TR_C**: Return period of the limit capacity seismic action for each of the limit states.
- **TR_D**: Spectral return period for each of the limit states.

The input of analysis and parameters groups to be checked is required through sensitivity analyses. A set of 6 groups of aleatory uncertainties(G) were considered.

For each group it is required to enter the parameters to be examined and its minimum and maximum values within which each parameter can vary.

- ALEATORY UNCERTAINTIES = 6 GROUPS

(G1) – Mechanical parameters of masonry

(It's a group that includes elastic Young modulus "E", shear modulus "G", masonry compressive strength "fm", masonry shear strength "to", specific weight of masonry "w")

(G2) – Intermediate diaphragms stiffness

(It includes shear modulus (G) of intermediate diaphragms)

(G3) – Roof diaphragms stiffness

(It includes shear modulus (G) of roof diaphragms)

(G4) – Staircases stiffness

(it includes shear modulus (G) of intermediate diaphragms)

(G5) – Constitutive laws of masonry panels

(in terms of shear-force drift and flexion-force drift)

(G6) – Vaults thickness

(it includes the different vaults thickness)

Epistemic uncertainties are usually related to constructive or modelling factors.

When the sensitivity class (SC) is higher and the data acquired are sufficient to assign a subjective probability related to the level of reliability of each choice, the combination through the logic tree approach is advisable.

On the contrary, when the final assessment is slightly affected by epistemic uncertainties, it's suggested to make a choice among the alternatives considered in order to limit the final computational effort.

In this case one epistemic uncertainty was considered.

The epistemic uncertainty associated with the effectiveness of wall-to-wall connections was found to be the most relevant.

<u>- EPISTEMIC UNCERTAINTIES = 1</u>

"flange effect" associated with the quality of wall to wall connections .

For the two lower analyses obtained by the preliminary modelling (one for the X direction and one for the Y direction), I performed nonlinear static analyses using the adjourned values.

This analysis underlines the importance of the single structural characteristics in the global behavior of the building.

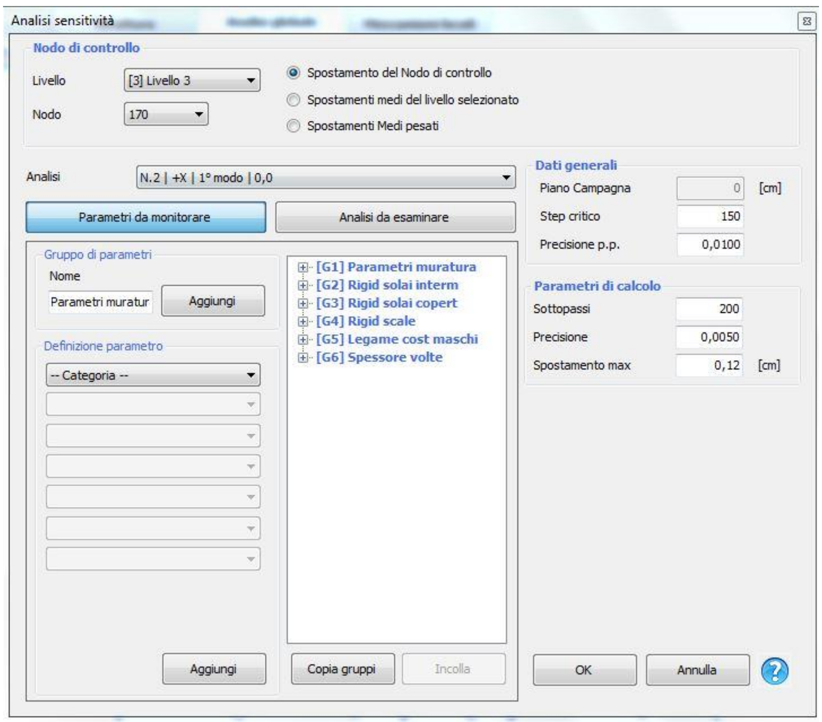


Figure 81. Analysis N.2 – "X" direction aleatory uncertainties

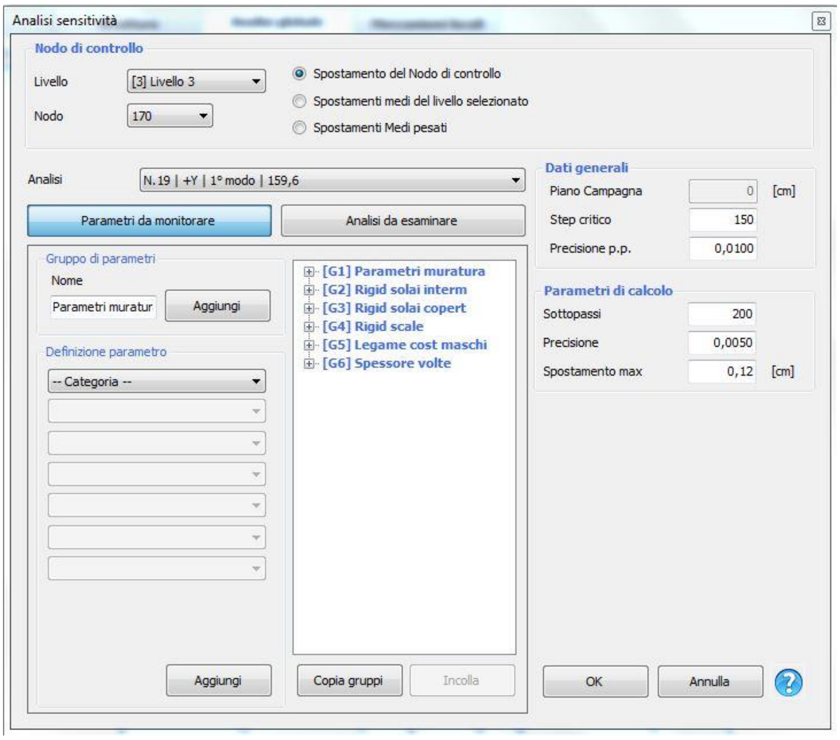


Figure 82. Analysis N.19 – "Y" direction aleatory uncertainties

ALEATORY UNCERTAINTIES: low, up and mean values				
Gk		**Gk,low**	**Gk,up**	**Gk,mean**
G1 - mechanical parameters	E(N/mm2)	1230	1500	1365
	G(N/mm2)	410	500	455
	fm(N/cm2)	250	320	285
	T0(N/cm2)	4,3	10,5	7,4
	w(KN/m3)	18	20	19
G2 - intermediate diaphragms stiffness	$G_{plane1,eq}$(N/mm2)	1250	12500	6875
	$G_{plane2,eq}$(N/mm2)	1250	12500	6875
	$G_{plane3,eq}$(N/mm2)	1250	12500	6875
	$G_{plane4,eq}$(N/mm2)	1250	12500	6875
	$G_{plane6,eq}$(N/mm2)	1250	12500	6875
	$G_{plane7,eq}$(N/mm2)	1250	12500	6875
	$G_{plane8,eq}$(N/mm2)	1250	12500	6875
	$G_{plane9,eq}$(N/mm2)	1250	12500	6875
	$G_{plane10,eq}$(N/mm2)	1250	12500	6875
	$G_{plane11,eq}$(N/mm2)	1250	12500	6875
	$G_{plane13,eq}$(N/mm2)	1250	12500	6875
	$G_{plane14,eq}$(N/mm2)	1250	12500	6875
	$G_{plane15,eq}$(N/mm2)	1250	12500	6875
	$G_{plane16,eq}$(N/mm2)	1250	12500	6875
	$G_{plane39,eq}$(N/mm2)	1250	12500	6875
	$G_{plane17,eq}$(N/mm2)	1250	12500	6875

	$G_{plane18,eq}$(N/mm2)	1250	12500	6875
	$G_{plane19,eq}$(N/mm2)	1250	12500	6875
	$G_{plane20,eq}$(N/mm2)	1250	12500	6875
	$G_{plane21,eq}$(N/mm2)	1250	12500	6875
	$G_{plane22,eq}$(N/mm2)	1250	12500	6875
	$G_{plane23,eq}$(N/mm2)	1250	12500	6875
	$G_{plane24,eq}$(N/mm2)	1250	12500	6875
	$G_{plane25,eq}$(N/mm2)	1250	12500	6875
	$G_{plane26,eq}$(N/mm2)	1250	12500	6875
	$G_{plane27,eq}$(N/mm2)	1250	12500	6875
	$G_{plane28,eq}$(N/mm2)	1250	12500	6875
	$G_{plane29,eq}$(N/mm2)	1250	12500	6875
	$G_{plane31,eq}$(N/mm2)	1250	12500	6875
	$G_{plane32,eq}$(N/mm2)	1250	12500	6875
	$G_{plane33,eq}$(N/mm2)	1250	12500	6875
	$G_{plane34,eq}$(N/mm2)	1250	12500	6875
	$G_{plane35,eq}$(N/mm2)	1250	12500	6875
	$G_{plane36,eq}$(N/mm2)	1250	12500	6875
	$G_{plane37,eq}$(N/mm2)	1250	12500	6875
	$G_{plane38,eq}$(N/mm2)	1250	12500	6875
G3 - roof diaphragms stiffness	$G_{plane40,eq}$(N/mm2)	100	1000	550
	$G_{plane41,eq}$(N/mm2)	100	1000	550
	$G_{plane42,eq}$(N/mm2)	100	1000	550
	$G_{plane43,eq}$(N/mm2)	100	1000	550
	$G_{plane44,eq}$(N/mm2)	100	1000	550
	$G_{plane45,eq}$(N/mm2)	100	1000	550
	$G_{plane46,eq}$(N/mm2)	100	1000	550
	$G_{plane47,eq}$(N/mm2)	100	1000	550
	$G_{plane48,eq}$(N/mm2)	100	1000	550
	$G_{plane49,eq}$(N/mm2)	100	1000	550
	$G_{plane50,eq}$(N/mm2)	100	1000	550
	$G_{plane51,eq}$(N/mm2)	100	1000	550
	$G_{plane52,eq}$(N/mm2)	100	1000	550
	$G_{plane53,eq}$(N/mm2)	100	1000	550
	$G_{plane54,eq}$(N/mm2)	100	1000	550
	$G_{plane55,eq}$(N/mm2)	100	1000	550
G4 - staircases stiffness	$G_{stair12,eq}$(N/mm2)	1250	12500	6875
G5 - constitutive laws of masonry panels	Shear-force drift(%)	0,006	0,009	0,0075
	Flexion-force drift(%)	0,012	0,018	0,015
G6 - vaults thickness	Vault1-thickness(cm)	15	40	27,5
	Vault2-thickness(cm)	15	40	27,5
	Vault3-thickness(cm)	15	40	27,5
	Vault4-thickness(cm)	15	40	27,5
	Vault5-thickness(cm)	15	40	27,5
	Vault6-thickness(cm)	15	40	27,5

	Vault7-thickness(cm)	15	40	27,5
	Vault8-thickness(cm)	15	40	27,5
	Vault9-thickness(cm)	15	40	27,5
	Vault10-thickness(cm)	15	40	27,5
	Vault11-thickness(cm)	15	40	27,5
	Vault12-thickness(cm)	15	40	27,5
	Vault13-thickness(cm)	15	40	27,5

Figure 83. Aleatory uncertainties: low, up and mean values

Once the sensitivity analyses were completed and all results post-processed, it is possible to proceed to the next step, that is the attribution of a Sensitivity Class (SC): high, medium and low sensitivity.

- ANALYSIS N.2 ("X" direction) – SLO

Descrizione	α PGA
[1] G1 + G2 + G3 + G4 + G5 + G6	2,247
[G1] Parametri muratura (0,21799 / -0,08622)	
[2] G1{min} + G2 + G3 + G4 + G5 + G6	1,757
[3] G1{max} + G2 + G3 + G4 + G5 + G6	2,441
[G2] Rigid solai interm (0,00456 / -0,00136)	
[4] G1 + G2{min} + G3 + G4 + G5 + G6	2,237
[5] G1 + G2{max} + G3 + G4 + G5 + G6	2,250
[G3] Rigid solai copert (0,01038 / -0,00401)	
[6] G1 + G2 + G3{min} + G4 + G5 + G6	2,256
[7] G1 + G2 + G3{max} + G4 + G5 + G6	2,224
[G4] Rigid scale (0,05823 / -0,00061)	
[8] G1 + G2 + G3 + G4{min} + G5 + G6	2,116
[9] G1 + G2 + G3 + G4{max} + G5 + G6	2,249
[G5] Legame cost maschi (0,16563 / 0,00000)	
[10] G1 + G2 + G3 + G4 + G5{min} + G6	1,875
[11] G1 + G2 + G3 + G4 + G5{max} + G6	2,185
[G6] Spessore volte (0,07503 / -0,01835)	
[12] G1 + G2 + G3 + G4 + G5 + G6{min}	2,288
[13] G1 + G2 + G3 + G4 + G5 + G6{max}	2,079

ATTRIBUTION OF THE SENSITIVITY CLASS (SC)	
Gk	SC
G1 - mechanical parameters	High
G2 - intermediate diaphragms stiffness	Low
G3 - roof diaphragms stiffness	Low
G4 - staircases stiffness	Low
G5 - constitutive laws of masonry panels	High
G6 - vaults thickness	Medium

$$\alpha_{medio} = \alpha_{[1]}$$

$$\alpha_{min} = min(\alpha_{medio}; \overbrace{\alpha_{[2]}; \alpha_{[3]}}^{[G1]})$$

$$I_s = \frac{\alpha_{medio} - \alpha_{min}}{\alpha_{medio}}$$

Figure 84. Analysis N.2 (X dir.) SLO – sensitivity results

- ANALYSIS N.2 (“X” direction) – SLD

Descrizione	α PGA
[1] G1 + G2 + G3 + G4 + G5 + G6	1,829
[G1] Parametri muratura (0,21677 / -0,08856)	
[2] G1{min} + G2 + G3 + G4 + G5 + G6	1,432
[3] G1{max} + G2 + G3 + G4 + G5 + G6	1,991
[G2] Rigid solai interm (0,00491 / -0,00039)	
[4] G1 + G2{min} + G3 + G4 + G5 + G6	1,820
[5] G1 + G2{max} + G3 + G4 + G5 + G6	1,830
[G3] Rigid solai copert (0,01062 / -0,00267)	
[6] G1 + G2 + G3{min} + G4 + G5 + G6	1,834
[7] G1 + G2 + G3{max} + G4 + G5 + G6	1,810
[G4] Rigid scale (0,05495 / -0,00059)	
[8] G1 + G2 + G3 + G4{min} + G5 + G6	1,728
[9] G1 + G2 + G3 + G4{max} + G5 + G6	1,830
[G5] Legame cost maschi (0,15589 / 0,00000)	
[10] G1 + G2 + G3 + G4 + G5{min} + G6	1,544
[11] G1 + G2 + G3 + G4 + G5{max} + G6	1,782
[G6] Spessore volte (0,07046 / -0,01719)	
[12] G1 + G2 + G3 + G4 + G5 + G6{min}	1,860
[13] G1 + G2 + G3 + G4 + G5 + G6{max}	1,700

ATTRIBUTION OF THE SENSITIVITY CLASS (SC)	
Gk	SC
G1 - mechanical parameters	High
G2 - intermediate diaphragms stiffness	Low
G3 - roof diaphragms stiffness	Low
G4 - staircases stiffness	Low
G5 - constitutive laws of masonrypanels	High
G6 - vaults thickness	Low

$$\alpha_{medio} = \alpha_{[1]}$$

$$\alpha_{min} = min(\alpha_{medio}; \overbrace{\alpha_{[2]}; \alpha_{[3]}}^{[G1]})$$

$$I_s = \frac{\alpha_{medio} - \alpha_{min}}{\alpha_{medio}}$$

Figure 85. Analysis N.2 (X dir.) SLD – sensitivity results

- ANALYSIS N.2 (“X” direction) – SLV

Descrizione	α PGA
[1] G1 + G2 + G3 + G4 + G5 + G6	0,879
[G1] Parametri muratura (0,07586 / -0,11243)	
[2] G1{min} + G2 + G3 + G4 + G5 + G6	0,812
[3] G1{max} + G2 + G3 + G4 + G5 + G6	0,977
[G2] Rigid solai interm (0,00000 / -0,02231)	
[4] G1 + G2{min} + G3 + G4 + G5 + G6	0,893
[5] G1 + G2{max} + G3 + G4 + G5 + G6	0,898
[G3] Rigid solai copert (0,00000 / -0,01864)	
[6] G1 + G2 + G3{min} + G4 + G5 + G6	0,895
[7] G1 + G2 + G3{max} + G4 + G5 + G6	0,890
[G4] Rigid scale (0,00472 / -0,00057)	
[8] G1 + G2 + G3 + G4{min} + G5 + G6	0,874
[9] G1 + G2 + G3 + G4{max} + G5 + G6	0,879
[G5] Legame cost maschi (0,07324 / -0,11252)	
[10] G1 + G2 + G3 + G4 + G5{min} + G6	0,814
[11] G1 + G2 + G3 + G4 + G5{max} + G6	0,977
[G6] Spessore volte (0,00000 / -0,01526)	
[12] G1 + G2 + G3 + G4 + G5 + G6{min}	0,892
[13] G1 + G2 + G3 + G4 + G5 + G6{max}	0,883

ATTRIBUTION OF THE SENSITIVITY CLASS (SC)	
Gk	SC
G1 - mechanical parameters	High
G2 - intermediate diaphragms stiffness	Low
G3 - roof diaphragms stiffness	Low
G4 - staircases stiffness	Low
G5 - constitutive laws of masonry panels	High
G6 - vaults thickness	Low

$$\alpha_{medio} = \alpha_{[1]}$$

$$\alpha_{min} = min(\alpha_{medio}; \overbrace{\alpha_{[2]}; \alpha_{[3]}}^{[G1]})$$

$$I_s = \frac{\alpha_{medio} - \alpha_{min}}{\alpha_{medio}}$$

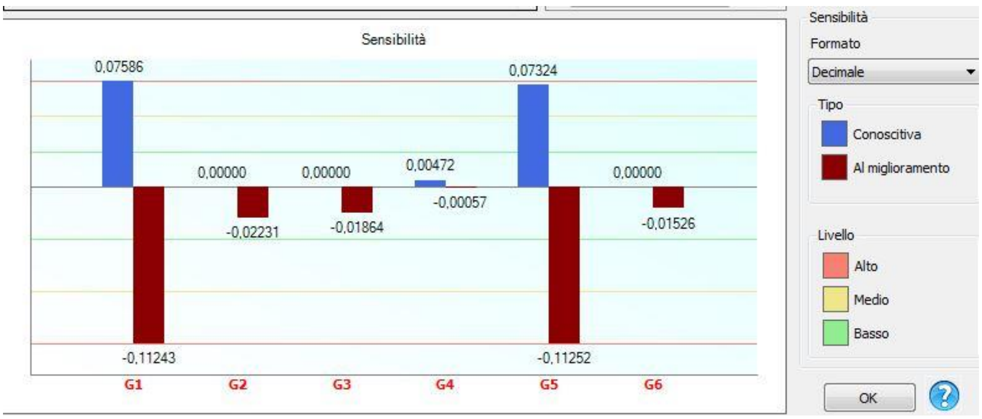


Figure 86. Analysis N.2 (X dir.) SLV – sensitivity results

- ANALYSIS N.19 (“Y” direction) – SLO

Descrizione	α PGA
[1] G1 + G2 + G3 + G4 + G5 + G6	1,813
[G1] Parametri muratura (0,09902 / -0,11228)	
[2] G1{min} + G2 + G3 + G4 + G5 + G6	1,634
[3] G1{max} + G2 + G3 + G4 + G5 + G6	2,017
[G2] Rigid solai interm (0,13048 / 0,00000)	
[4] G1 + G2{min} + G3 + G4 + G5 + G6	1,576
[5] G1 + G2{max} + G3 + G4 + G5 + G6	1,733
[G3] Rigid solai copert (0,01671 / -0,04706)	
[6] G1 + G2 + G3{min} + G4 + G5 + G6	1,783
[7] G1 + G2 + G3{max} + G4 + G5 + G6	1,898
[G4] Rigid scale (0,00298 / -0,00025)	
[8] G1 + G2 + G3 + G4{min} + G5 + G6	1,808
[9] G1 + G2 + G3 + G4{max} + G5 + G6	1,813
[G5] Legame cost maschi (0,10243 / -0,13509)	
[10] G1 + G2 + G3 + G4 + G5{min} + G6	1,627
[11] G1 + G2 + G3 + G4 + G5{max} + G6	2,058
[G6] Spessore volte (0,00000 / -0,09561)	
[12] G1 + G2 + G3 + G4 + G5 + G6{min}	1,833
[13] G1 + G2 + G3 + G4 + G5 + G6{max}	1,986

ATTRIBUTION OF THE SENSITIVITY CLASS (SC)	
Gk	SC
G1 - mechanical parameters	High
G2 - intermediate diaphragms stiffness	High
G3 - roof diaphragms stiffness	Low
G4 - staircases stiffness	Low
G5 - constitutive laws of masonry panels	High
G6 - vaults thickness	Low

$$\alpha_{medio} = \alpha_{[1]}$$

$$\alpha_{min} = min(\alpha_{medio}; \overbrace{\alpha_{[2]}; \alpha_{[3]}}^{[G1]})$$

$$I_s = \frac{\alpha_{medio} - \alpha_{min}}{\alpha_{medio}}$$

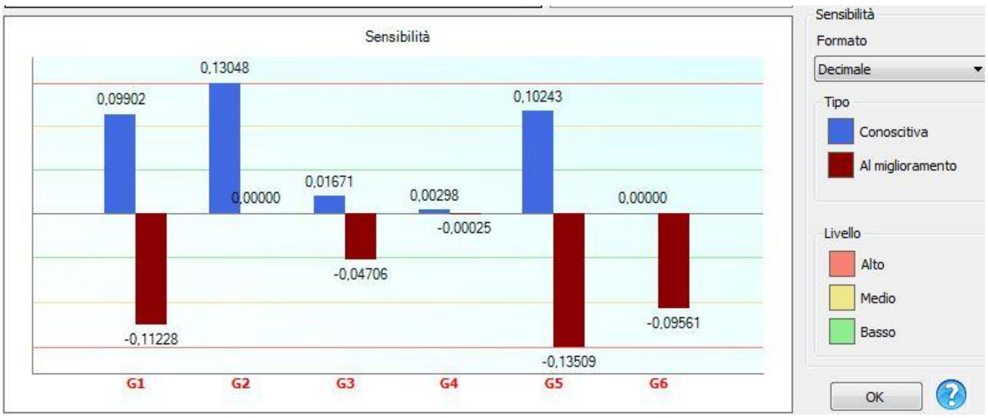


Figure 87. Analysis N.19 (Y dir.) SLO – sensitivity results

- ANALYSIS N.19 ("Y" direction) – SLD

Descrizione	α PGA
[1] G1 + G2 + G3 + G4 + G5 + G6	1,471
[G1] Parametri muratura (0,10352 / -0,11157)	
[2] G1{min} + G2 + G3 + G4 + G5 + G6	1,319
[3] G1{max} + G2 + G3 + G4 + G5 + G6	1,636
[G2] Rigid solai interm (0,12856 / 0,00000)	
[4] G1 + G2{min} + G3 + G4 + G5 + G6	1,282
[5] G1 + G2{max} + G3 + G4 + G5 + G6	1,412
[G3] Rigid solai copert (0,01400 / -0,04533)	
[6] G1 + G2 + G3{min} + G4 + G5 + G6	1,451
[7] G1 + G2 + G3{max} + G4 + G5 + G6	1,538
[G4] Rigid scale (0,00330 / -0,00028)	
[8] G1 + G2 + G3 + G4{min} + G5 + G6	1,467
[9] G1 + G2 + G3 + G4{max} + G5 + G6	1,472
[G5] Legame cost maschi (0,10025 / -0,12856)	
[10] G1 + G2 + G3 + G4 + G5{min} + G6	1,324
[11] G1 + G2 + G3 + G4 + G5{max} + G6	1,661
[G6] Spessore volte (0,00000 / -0,09103)	
[12] G1 + G2 + G3 + G4 + G5 + G6{min}	1,488
[13] G1 + G2 + G3 + G4 + G5 + G6{max}	1,605

ATTRIBUTION OF THE SENSITIVITY CLASS (SC)	
Gk	SC
G1 - mechanical parameters	High
G2 - intermediate diaphragms stiffness	High
G3 - roof diaphragms stiffness	Low
G4 - staircases stiffness	Low
G5 - constitutive laws of masonry panels	High
G6 - vaults thickness	Low

$$\alpha_{medio} = \alpha_{[1]}$$

$$\alpha_{min} = min(\alpha_{medio}; \overbrace{\alpha_{[2]}; \alpha_{[3]}}^{[G1]})$$

$$I_s = \frac{\alpha_{medio} - \alpha_{min}}{\alpha_{medio}}$$

Figure 88. Analysis N.19 (Y dir.) SLD – sensitivity results

- ANALYSIS N.19 (“Y” direction) – SLV

Descrizione	a PGA
[1] G1 + G2 + G3 + G4 + G5 + G6	0,793
[G1] Parametri muratura (0,13153 / -0,10055)	
[2] G1{min} + G2 + G3 + G4 + G5 + G6	0,688
[3] G1{max} + G2 + G3 + G4 + G5 + G6	0,872
[G2] Rigid solai interm (0,00069 / -0,07620)	
[4] G1 + G2{min} + G3 + G4 + G5 + G6	0,853
[5] G1 + G2{max} + G3 + G4 + G5 + G6	0,792
[G3] Rigid solai copert (0,00603 / -0,00412)	
[6] G1 + G2 + G3{min} + G4 + G5 + G6	0,788
[7] G1 + G2 + G3{max} + G4 + G5 + G6	0,796
[G4] Rigid scale (0,00311 / -0,00025)	
[8] G1 + G2 + G3 + G4{min} + G5 + G6	0,790
[9] G1 + G2 + G3 + G4{max} + G5 + G6	0,793
[G5] Legame cost maschi (0,15001 / -0,17010)	
[10] G1 + G2 + G3 + G4 + G5{min} + G6	0,674
[11] G1 + G2 + G3 + G4 + G5{max} + G6	0,928
[G6] Spessore volte (0,02217 / -0,00862)	
[12] G1 + G2 + G3 + G4 + G5 + G6{min}	0,775
[13] G1 + G2 + G3 + G4 + G5 + G6{max}	0,800

ATTRIBUTION OF THE SENSITIVITY CLASS (SC)	
Gk	SC
G1 - mechanical parameters	High
G2 - intermediate diaphragms stiffness	Low
G3 - roof diaphragms stiffness	Low
G4 - staircases stiffness	Low
G5 - constitutive laws of masonry panels	High
G6 - vaults thickness	Low

$$\alpha_{medio} = \alpha_{[1]}$$

$$\alpha_{min} = min(\alpha_{medio}; \overbrace{\alpha_{[2]}; \alpha_{[3]}}^{[G1]})$$

$$I_s = \frac{\alpha_{medio} - \alpha_{min}}{\alpha_{medio}}$$

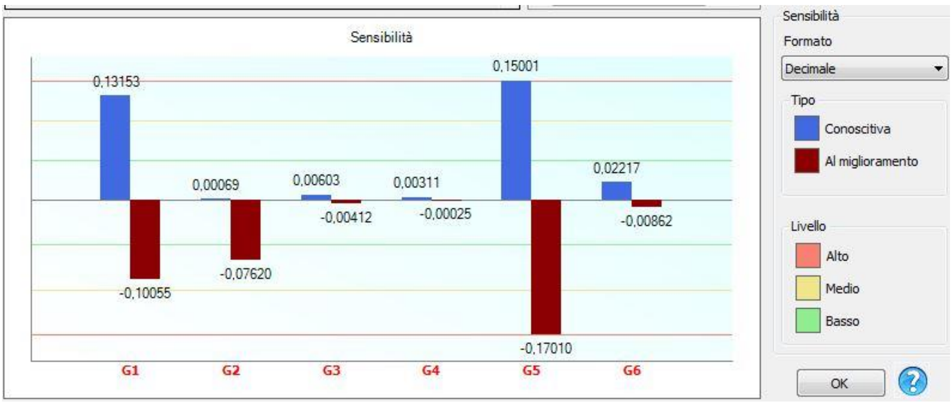


Figure 89. Analysis N.19 (Y dir.) SLV – sensitivity results

Among the aleatory variables, the mechanical properties of masonry-“G1” and constitutive laws of masonry panels-“G5”, were the most recurrent parameters associated with a high sensitivity class.

4.9. Definition of the plane of investigations and testing

Results from the sensitivity analysis are useful to optimize and reliably plan investigations and tests to be performed.

Indeed the objective of defining sensitivity classes is to identify the need for more investigation for the parameters that most significantly affect the seismic performance of the building.

Collaborating with Sibillina Dimora Srl company and with its staff, I was able to perform non-destructive tests and minor destructive tests on Palazzo Boldi.

DEFINITION OF THE PLAN OF INVESTIGATION		
Gk	**LCk**	**TEST TO BE PERFORMED**
G1 - mechanical parameters	(H)H	THE MECHANICAL PROPERTIES OF MASONRY REVEALED A RECURRING PARAMETER ASSOCIATED WITH A HIGH "SC", IT IS THEREFORE POSSIBLE TO PERFORME A MINOR DESTRUCTIVE TEST LIKE THE DOUBLE FLAT JACKS TEST, IN ORDER TO OBTAIN REALISTIC MECHANICAL PARAMETERS.
G2 - intermediate diaphragms stiffness	(M)H	DESPITE THE MEDIUM SENSITIVITY I CHOSE TO STILL OPT FOR A HIGH LEVEL OF KNOWLEDGE, THE TESTS TO BE PERFORMED TO DETERMINE THE STRATIGRAPHY OF THE FLOORS ARE CHEAP, EASY TO PERFORM AND NON-DESTRUCTIVE.
G3 - roof diaphragms stiffness	(L)H	IN THIS CASE I CHOSE TO REACH AN HIGH LEVEL OF KNOWLEDGE, THE TESTS TO BE PERFORMED TO KNOW THE STRATIGRAPHY OF THE ROOF ARE CHEAP, EASY TO PERFORM AND NON-DESTRUCTIVE.
G4 - staircases stiffness	(L)L	NO TEST WAS DONE.
G5 - constitutive laws of masonry panels	(H)L	NO TEST WAS DONE, THE EXECUTION OF OVERLY INVASIVE TESTS ON THE HISTORIC BUILDING ARE NOT PERMITTED.
G6 - vaults thickness	(L)H	IN THIS CASE I HAVE CHOSEN TO REACH A HIGH LEVEL OF KNOWLEDGE, THE TEST TO BE PERFORMED TO DETERMINE THE STRATIGRAPHY OF THE VAULTS ARE CHEAP, EASY TO PERFORM AND NON-DESTRUCTIVE.

Figure 90. Knowledge level (LC) to be achieved

<u>ALEATORY UNCERTAINTIES</u>

<u>G1 - Mechanical parameters of masonry – DOUBLE FLAT JACKS</u>

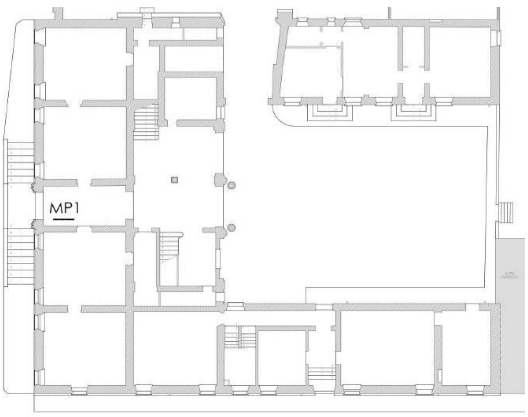
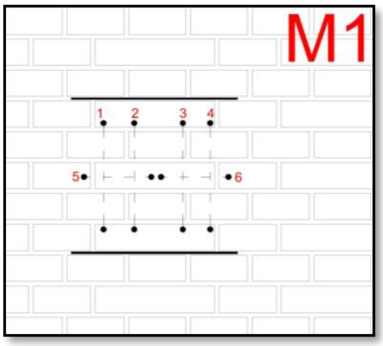


Figure 91. Ground floor plan - double flat jacks test



Tipo	Profondità (mm)	H provino (mm)	Lunghezza (mm)	Spessore (mm)	Area Lorda (mmq)	Pressione Martinetto (bar)	Area Eff (mmq)	Rapporto Aje/Ajg (Km)
MP350260	350	260	260	4	77865	5-40	72700	0,91

DATI INPUT							
Pressione		Base 1	Base 2	Base 3	Base 4	Base 5	Base 6
Basi di misurazioni	0	8,463	8,126	7,360	7,358	4,363	7,289
Taglio	0	8,871	8,759	7,491	7,733	4,810	7,530
1	1	8,547	8,140	6,919	7,510	4,781	7,445
	0	8,645	8,240	7,034	7,814	4,797	7,493
2	2	8,500	7,871	6,557	7,874	4,945	7,487
	0	8,638	7,942	6,713	7,671	4,899	7,536
3	3	8,485	7,738	6,271	7,683	5,106	7,528
	0	8,789	8,155	6,931	7,929	4,972	7,502
4	4	8,354	7,724	5,586	7,849	5,357	7,621
	0	8,712	8,198	6,512	8,154	5,180	7,498

Figure 92. Double flat jacks test

Figure 93. Double flat jacks test

Figure 94. Double flat jacks test

Figure 95. Double flat jacks test

Pressione		Pressione Effettiva σ	Base 1	Base 2	Base 3	Base 4	Media 1-2-3-4	Deformazione e	Base 5	Base 6	Media 5-6	Deformazione e
0	0		8,463	8,126	7,360	7,358	7,827		4,363	7,289	5,826	
T	0		8,871	8,759	7,491	7,733	8,214	-1,0743E-03	4,810	7,530	6,170	-6,6154E-04
1	2	1,82	8,547	8,140	6,919	7,510	7,779	1,3264E-04	4,781	7,445	6,113	-5,5192E-04
	0	0	8,645	8,240	7,034	7,814	7,933	-2,9583E-04	4,797	7,493	6,145	-6,1346E-04
2	4	3,64	8,500	7,871	6,557	7,874	7,701	3,5069E-04	4,945	7,487	6,216	-7,5000E-04
	0	0	8,638	7,942	6,713	7,671	7,741	2,3819E-04	4,899	7,536	6,218	-7,5288E-04
3	6	5,46	8,485	7,738	6,271	7,683	7,544	7,8472E-04	5,106	7,528	6,317	-9,4423E-04
	0	0	8,789	8,155	6,931	7,929	7,951	-3,4514E-04	4,972	7,502	6,237	-7,9038E-04
4	8	7,28	8,354	7,724	5,586	7,849	7,378	1,2458E-03	5,357	7,621	6,489	-1,2750E-03
	0	0	8,712	8,198	6,512	8,154	7,894	-1,8681E-04	5,180	7,498	6,339	-9,8654E-04

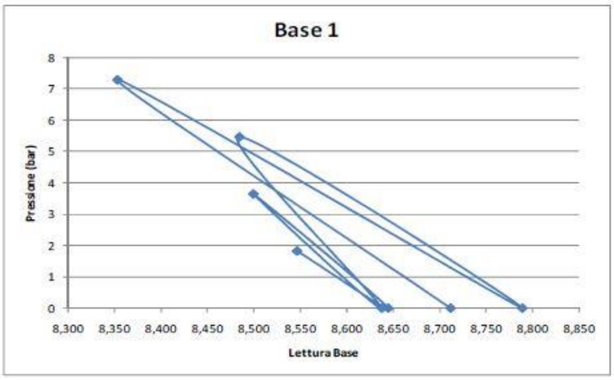
Figure 96. Results

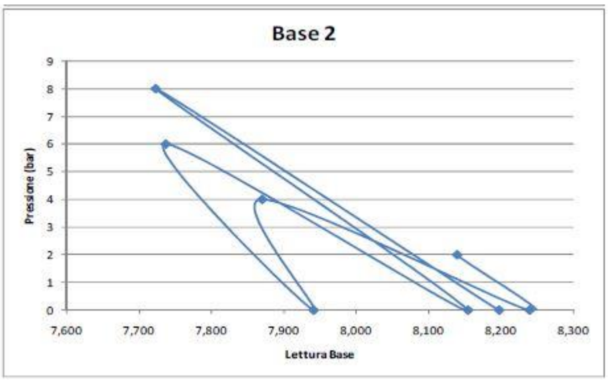
The results of the investigation were influenced by a breakup of a pipe during the tests.

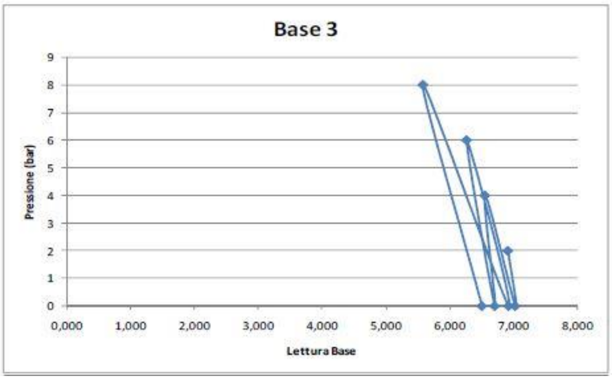
However, at the end, the final values of mechanical parameters of masonry were confirmed to be normative.

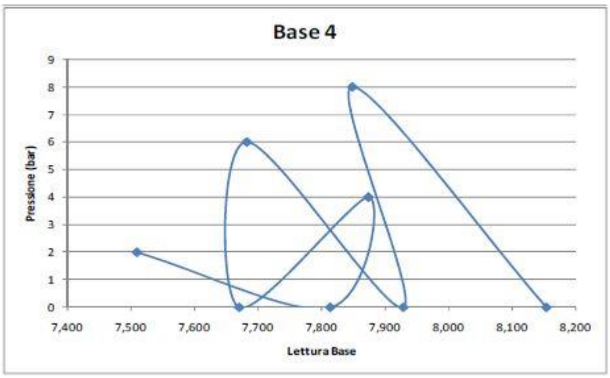
Masonry type	fm (N/cmq)	τ_0 (N/cmq)	E (N/mmq)	G (N/mmq)	W (kN/mc)
	min - max	min - max	min - max	min - max	
Masonry in bricks and lime mortar	240	6	1200	400	18
	400	9,2	1800	600	

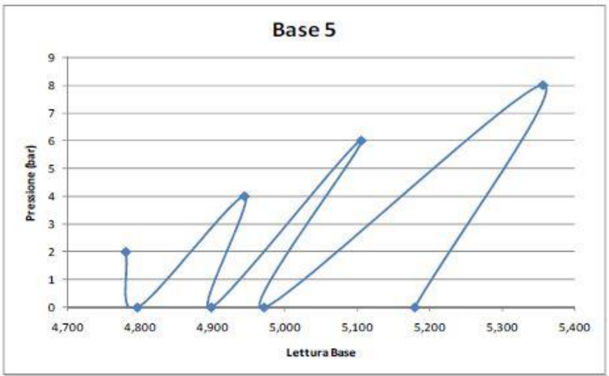
Figure 97. Masonry parameters

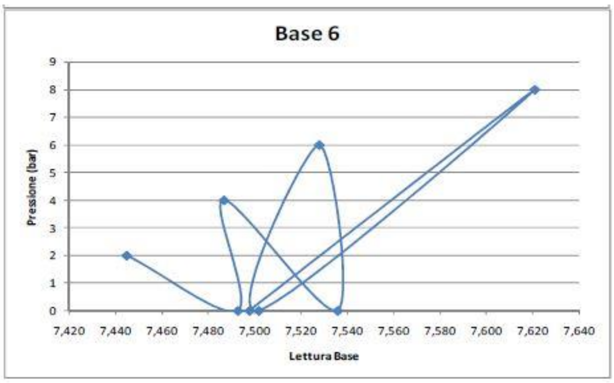












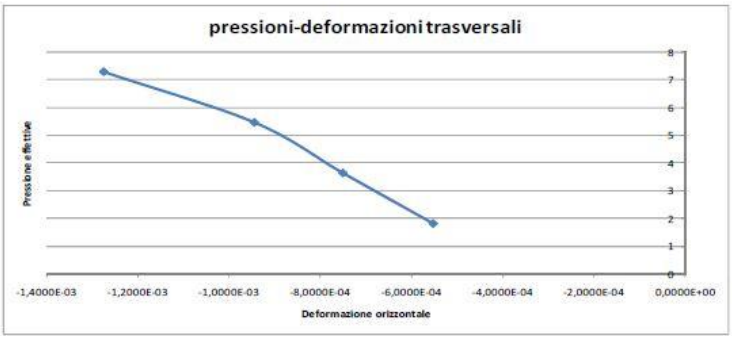


Figure 98. Results

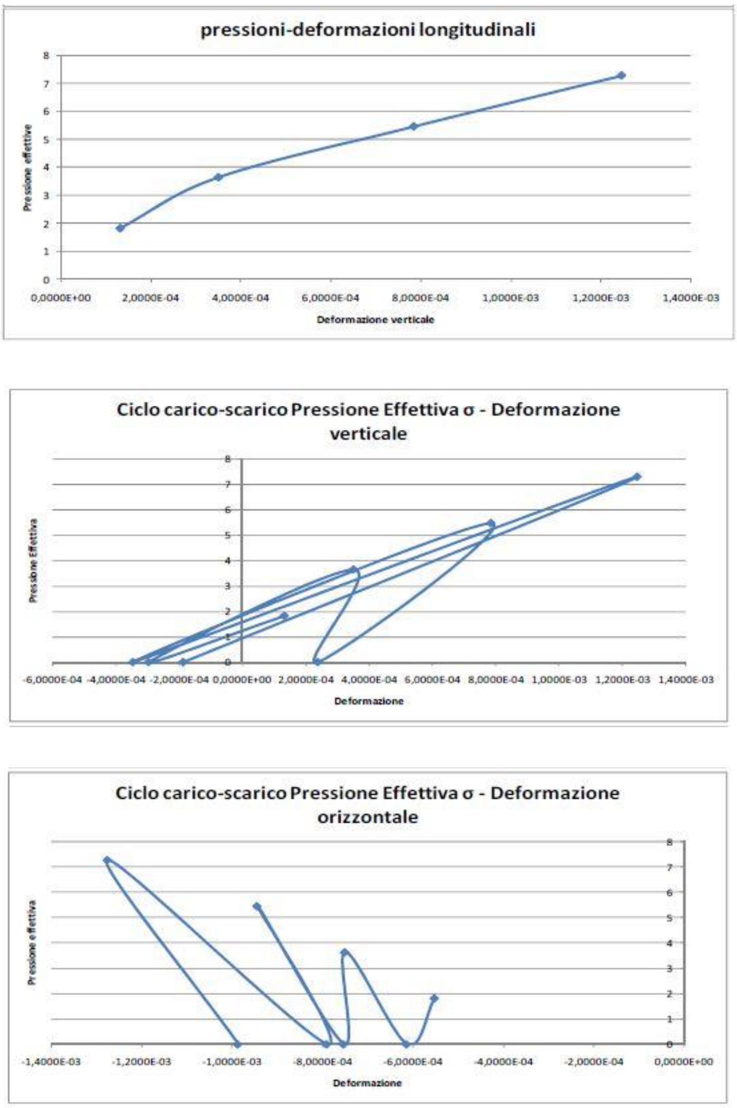


Figure 99. Results

G1 - Mechanical parameters of masonry – VISUAL/OPTICAL TESTING

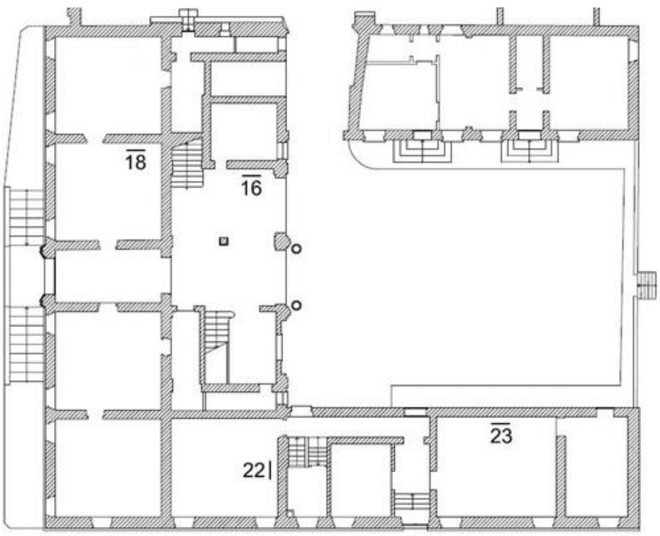


Figure 100. Ground floor plan – visual testing

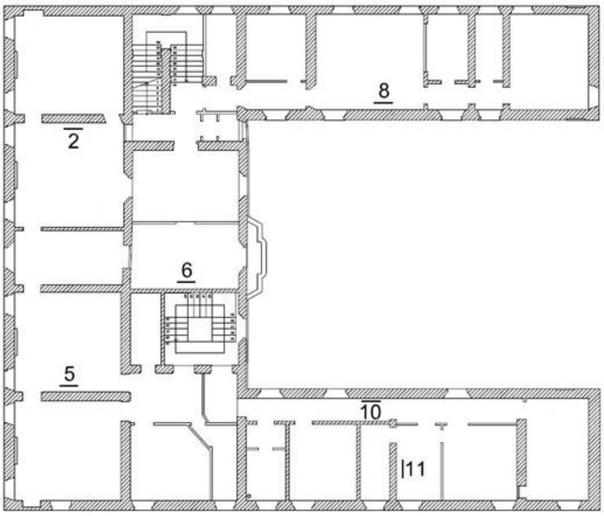


Figure 101. First floor plan – visual testing

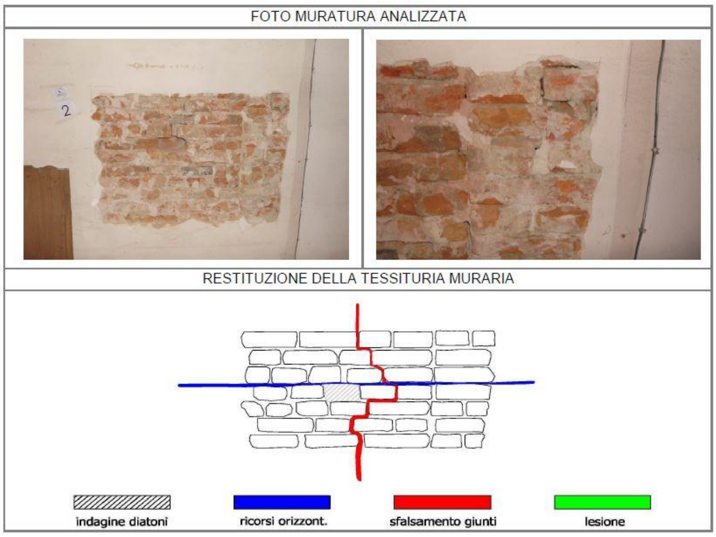


Figure 102. Investigation 1

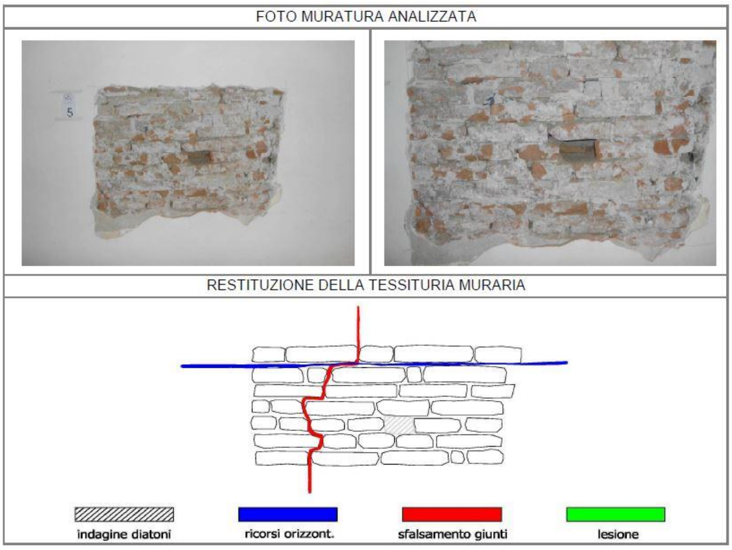


Figure 103. Investigation 5

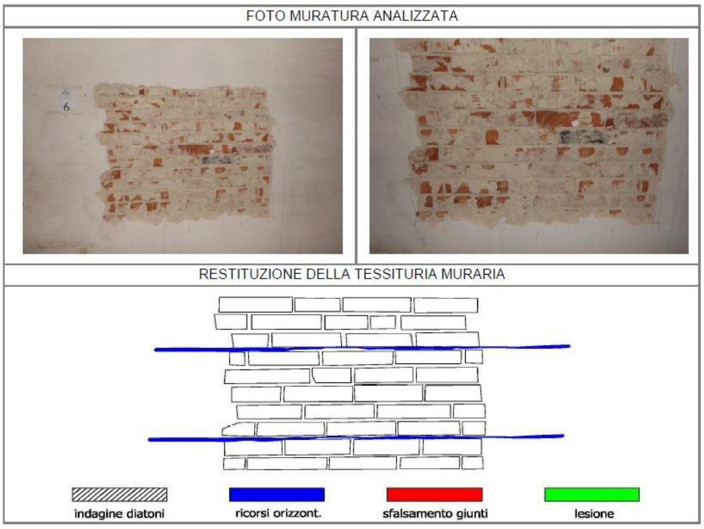


Figure 104. Investigation 6

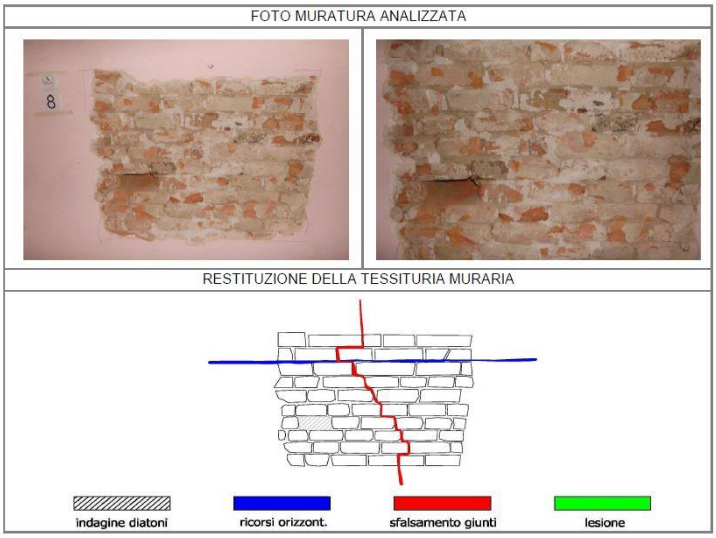


Figure 105. Investigation 8

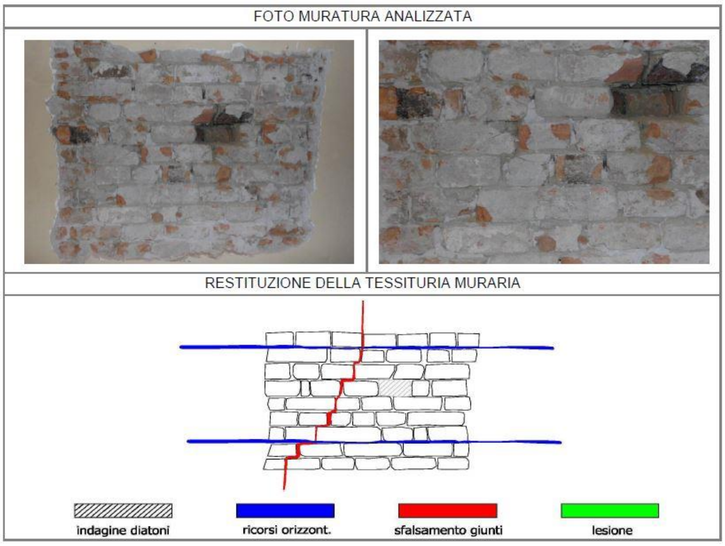


Figure 106. Investigation 10

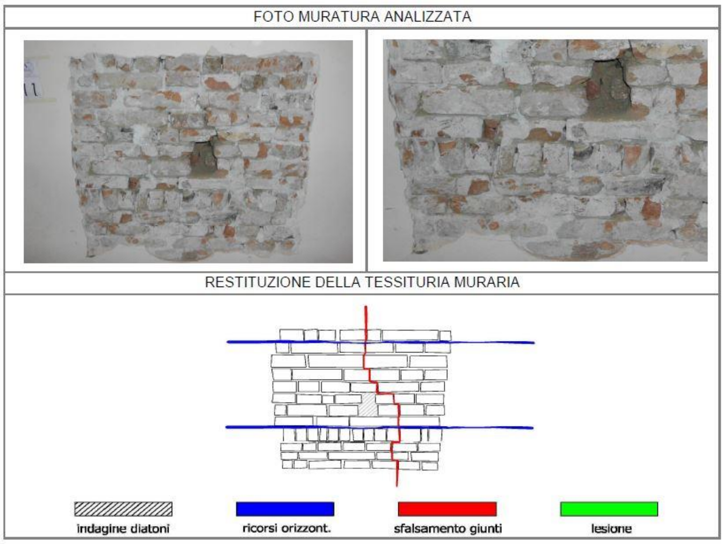


Figure 107. Investigation 11

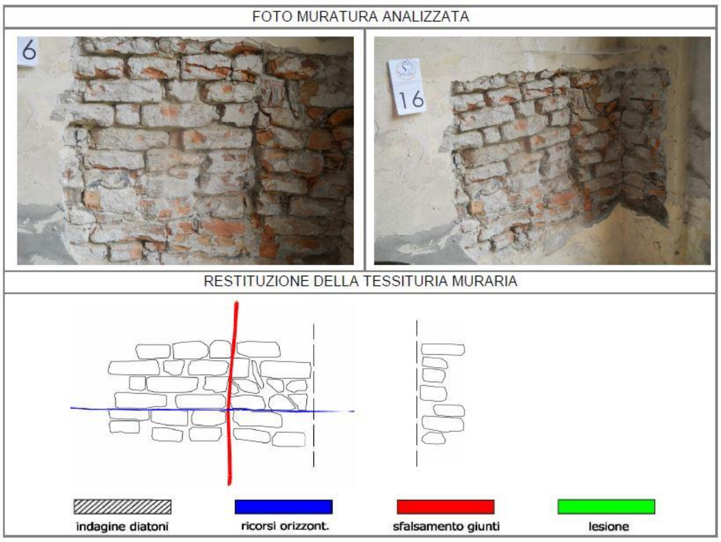


Figure 108. Investigation 16

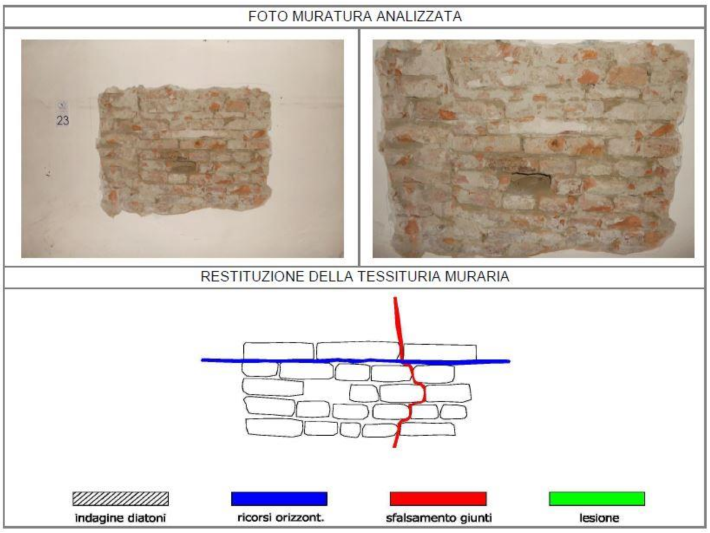


Figure 109. Investigation 23

G2 - Intermediate diaphragms stiffness – VISUAL/OPTICAL TESTING

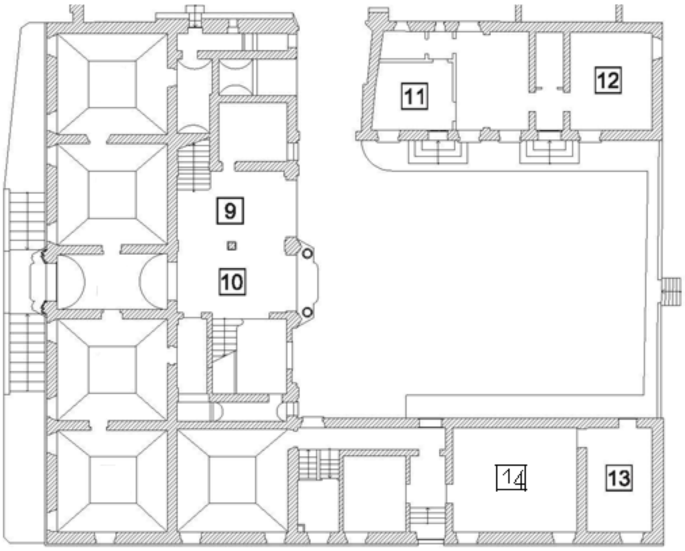


Figure 110. Ground floor plan – visual testing

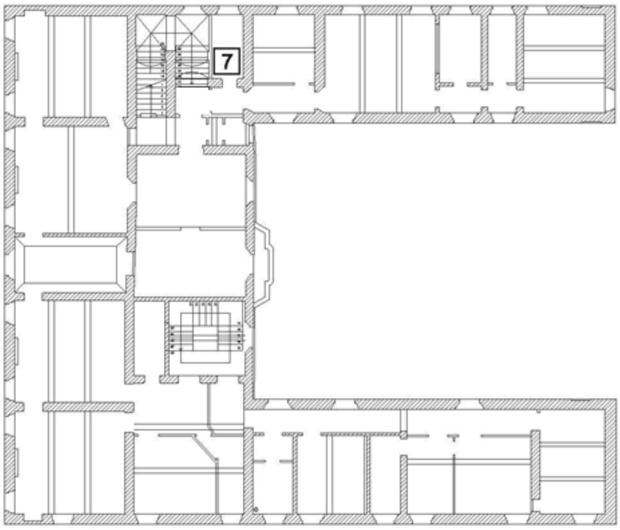
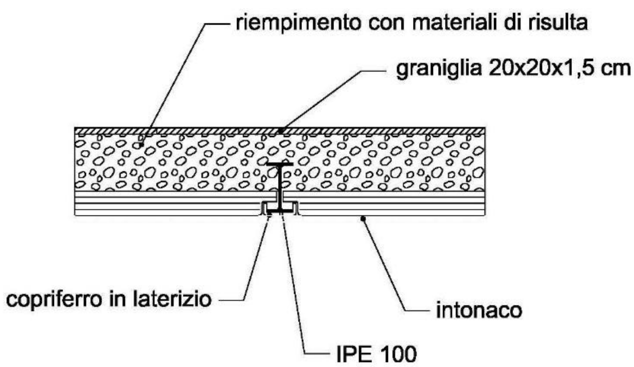


Figure 111. First floor plan – visual testing



Figures 112 - 113. Investigation 7 - stratigraphy

Figure 114. Investigation 8

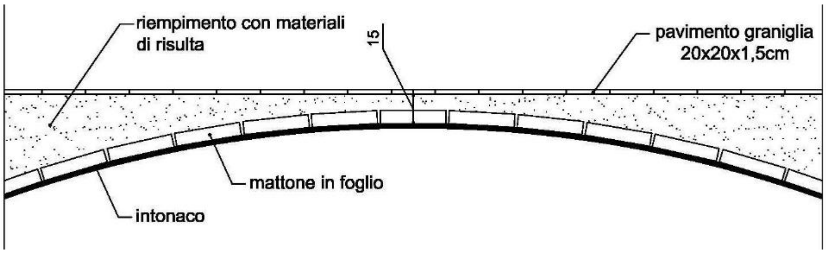


Figure 115. Investigation 8 – stratigraphy

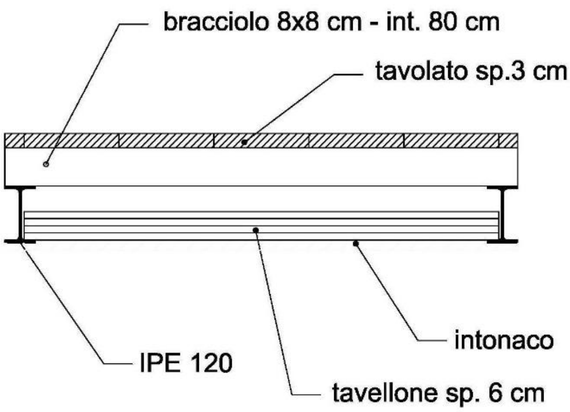
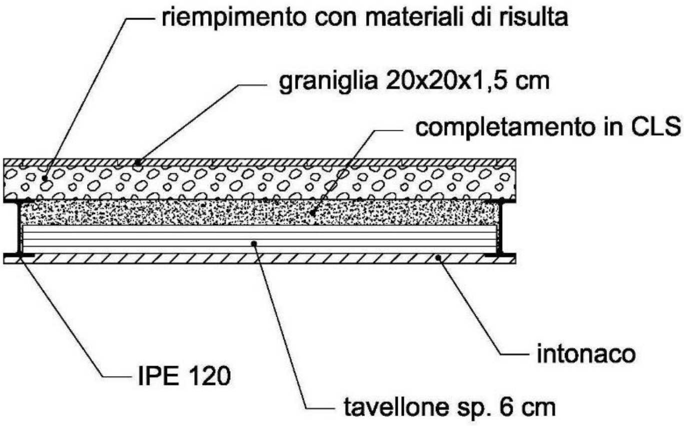


Figure 116-117. Investigation 9 - stratigraphy



Figures 118-119. Investigation 10 - stratigraphy

Figure 120. Investigation 11

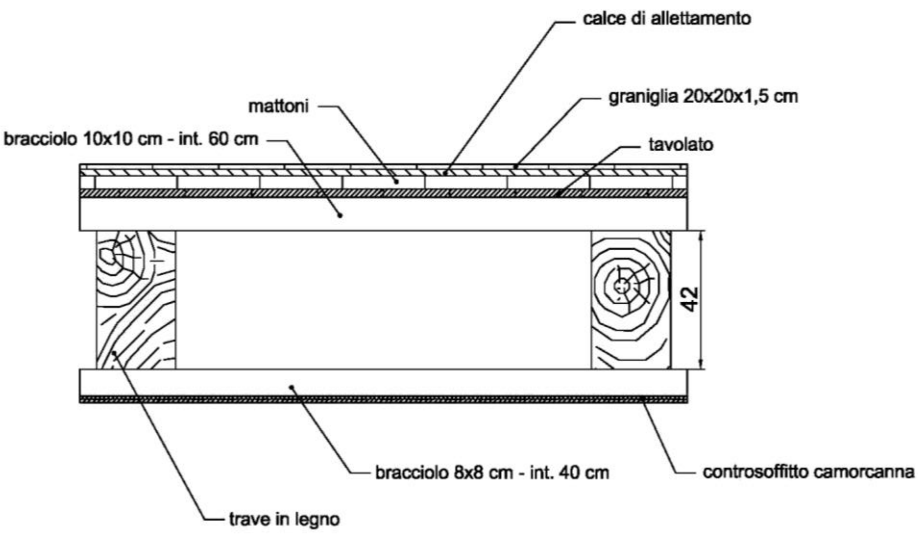


Figure 121. Investigation 11 - stratigraphy

Figure 122. Investigation 12

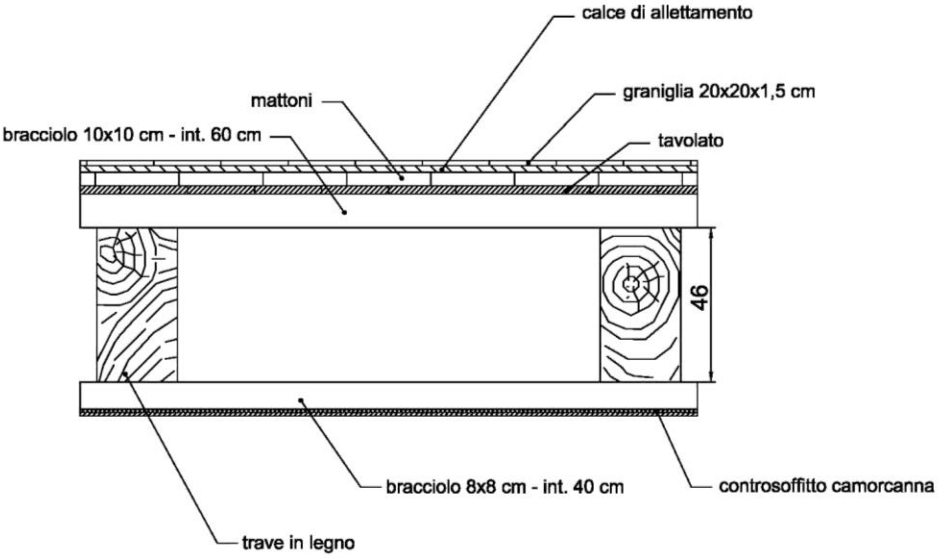
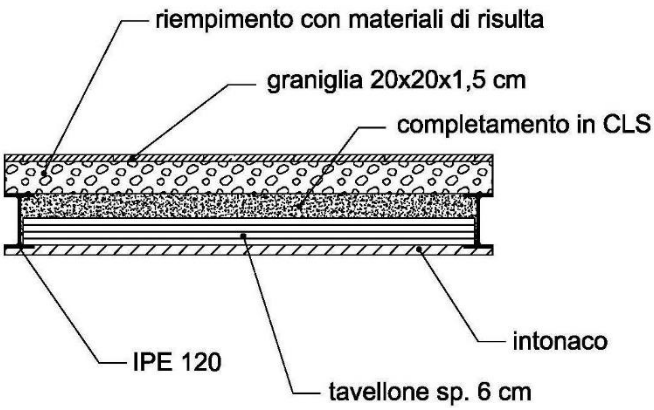


Figure 123. Investigation 12 - stratigraphy



Figures 124-125. Investigation 13 - stratigraphy

Figure 126. Investigation 14

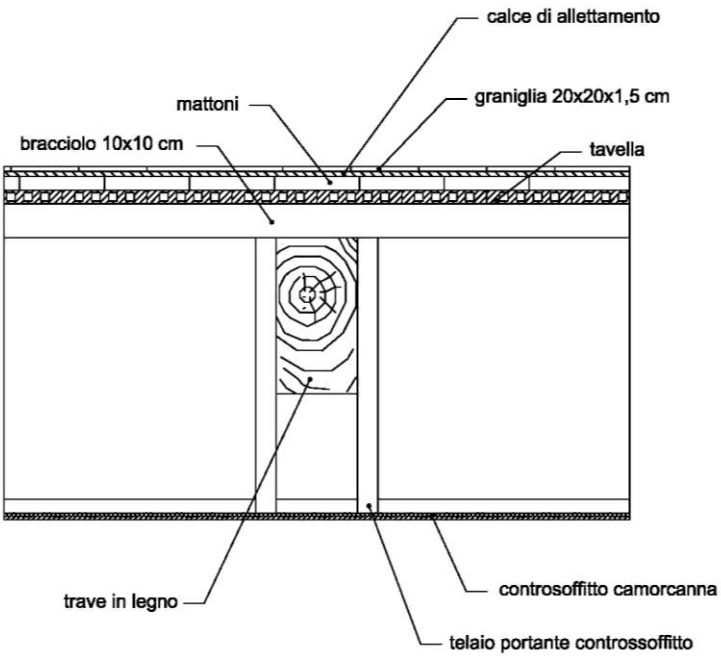


Figure 127. Investigation 14 - stratigraphy

G3 - Roof diaphragms stiffness – VISUAL/OPTICAL TESTING

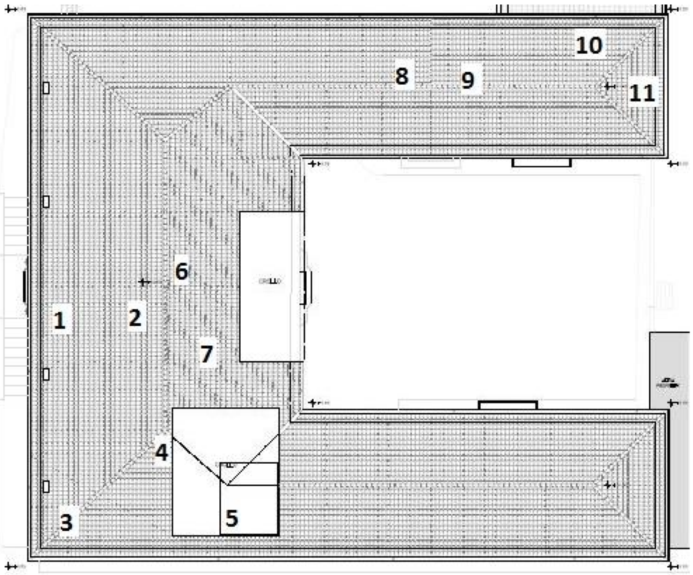


Figure 128. Roof plan – visual testing

Figure 129. Investigation 1

Figure 130. Investigation 2

Figure 131. Investigation 3

Figure 132. Investigation 4

Figure 133. Investigation 5

Figure 134. Investigation 6

Figure 135. Investigation 7

Figure 136. Investigation 8

Figure 137. Investigation 9

Figure 138. Investigation 10

Figure 139. Investigation 11

G4 – Staircases stiffness

The group “G4 - stiffness of staircases” has a low sensibility, no test was done.

G5 – Constitutive laws of masonry panels

More detailed information could be acquired only conducting destructive tests.

The execution of overly invasive tests on the historic building are not permitted.

No test was done.

G6 – Vaults thickness – VISUAL/OPTICAL TESTING

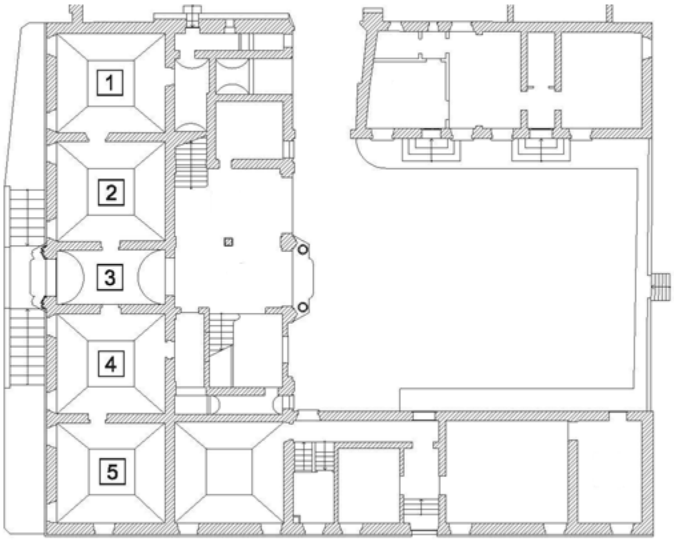


Figure 140. Ground floor plan – visual testing

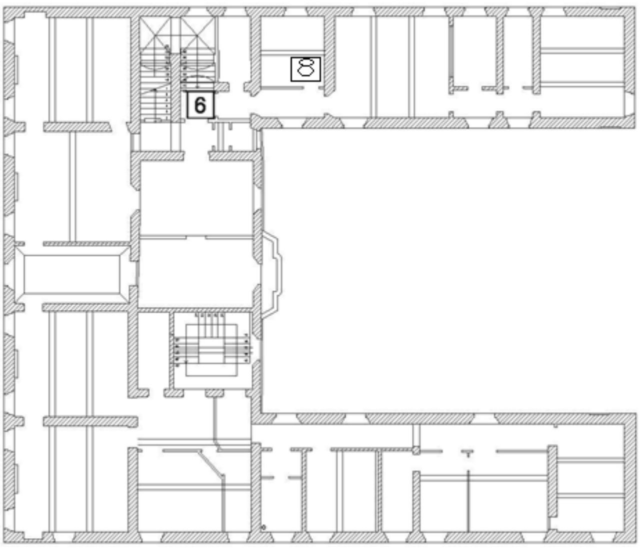


Figure 141. First floor plan – visual testing

Figure 142. Investigation 1

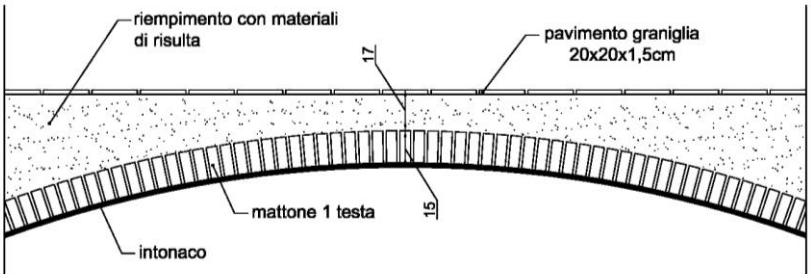


Figure 143. Investigation 1 – stratigraphy

Figure 144. Investigation 2

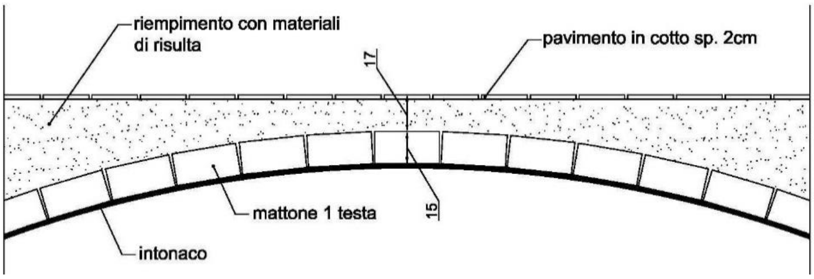


Figure 145. Investigation 2 - stratigraphy

Figure 146. Investigation 3

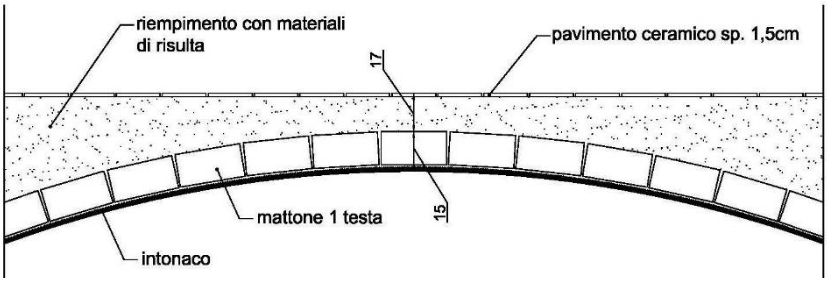


Figure 147. Investigation 3 - stratigraphy

Figure 148. Investigation n.4

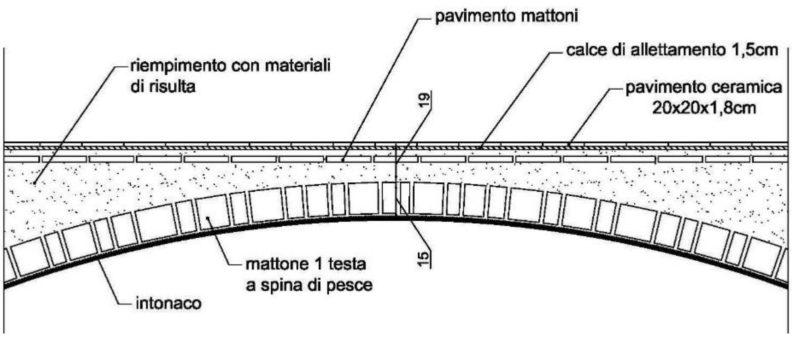


Figure 149. Investigation 4 - stratigraphy

Figure 150. Investigation n.5

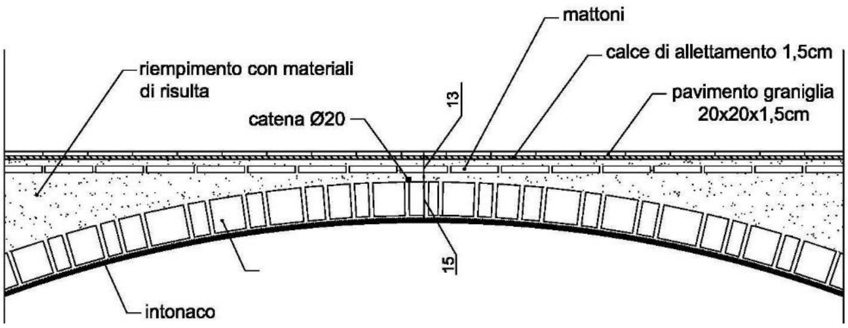


Figure 151. Investigation 5 - stratigraphy

Figure 152. Investigation n.6

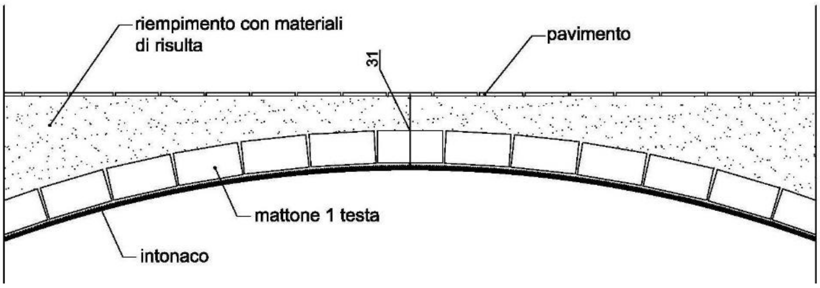


Figure 153. Investigation 6 - stratigraphy

Figure 154. Investigation n.8

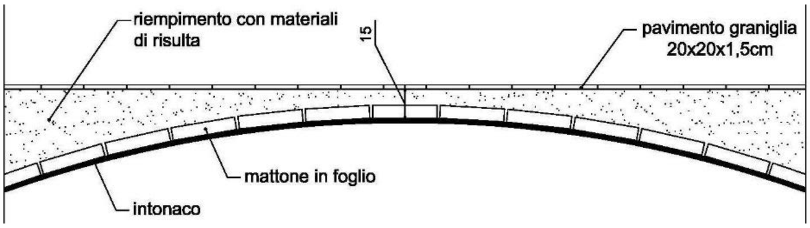


Figure 155. Investigation 8 - stratigraphy

EPISTEMIC UNCERTAINTY

"Flange effect associated with quality of the connections" – VISUAL/OPTICAL TESTING

Visual/optical testing on the connections were performed to acquire enough data to choose the most suitable model.

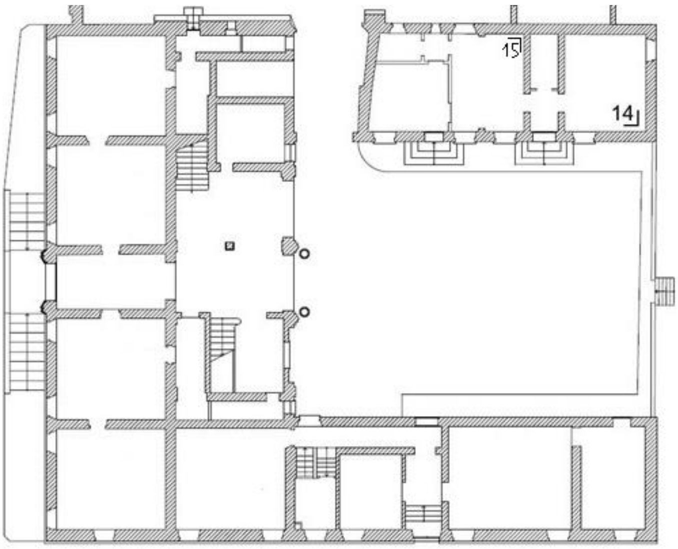


Figure 156. Ground floor plan – visual testing

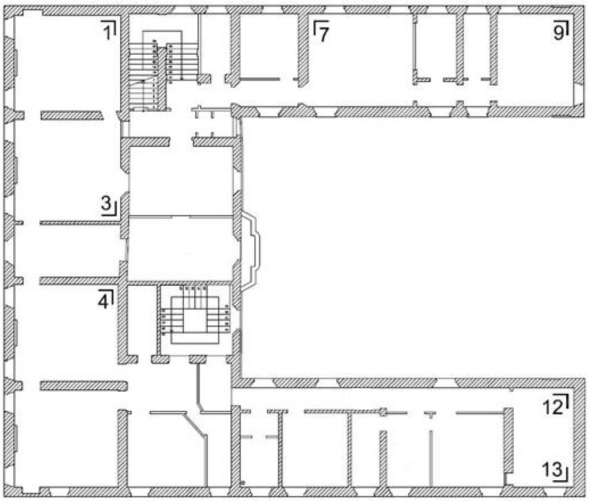


Figure 157. First floor plan – visual testing

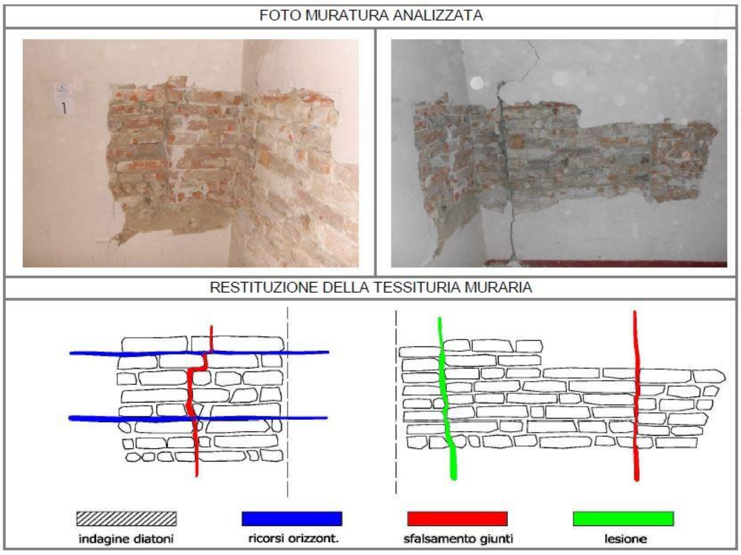


Figure 158. Investigation 1

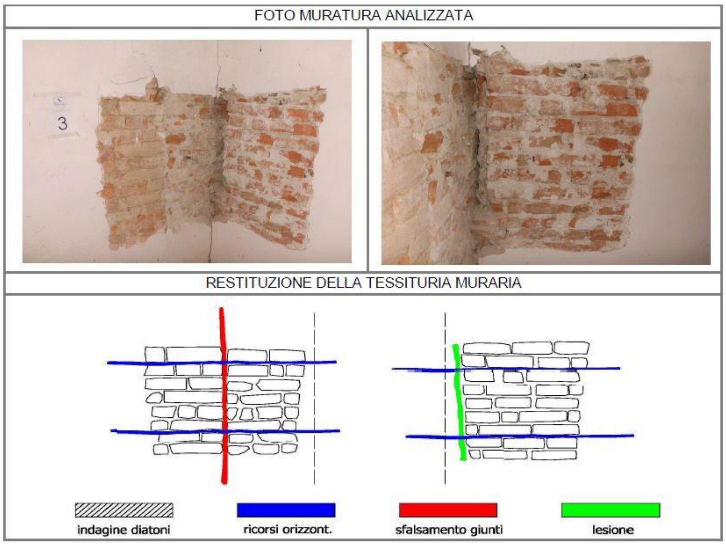


Figure 159. Investigation 3

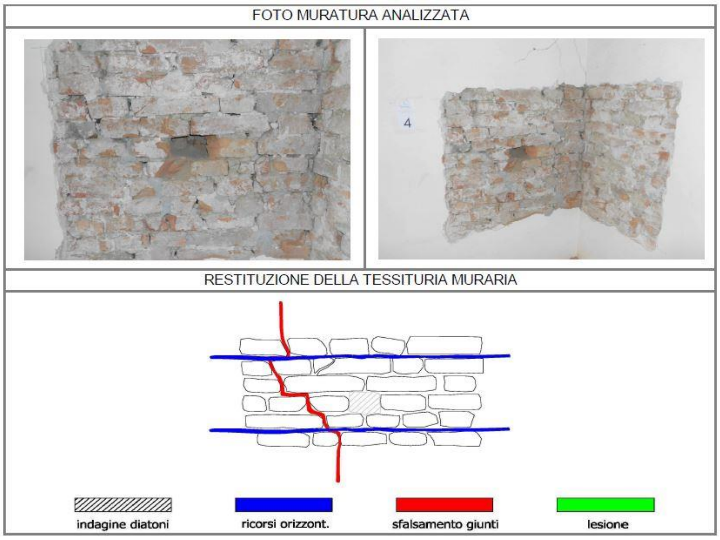


Figure 160. Investigation 4

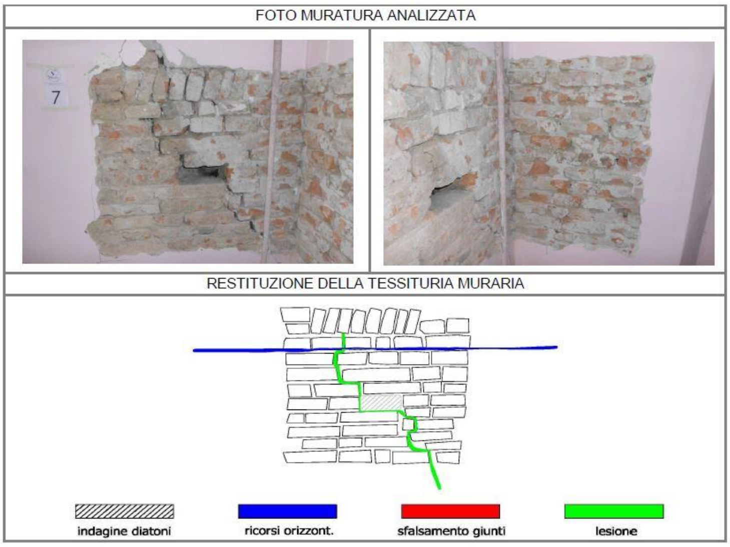


Figure 161. Investigation 7

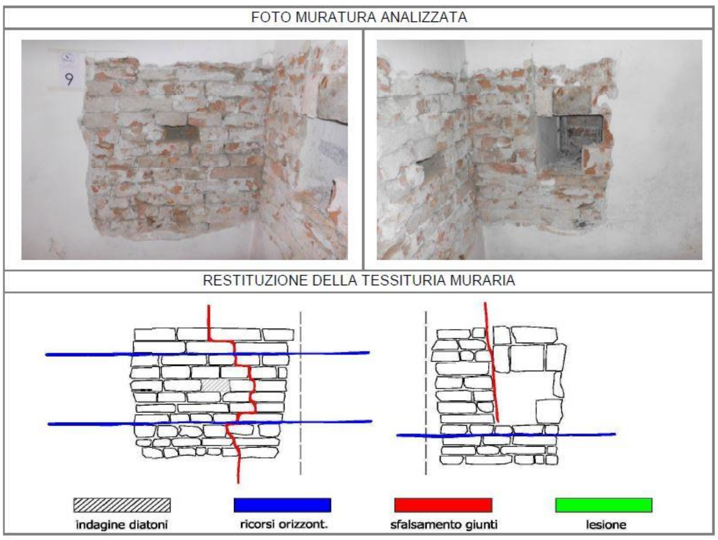


Figure 162. Investigation 9

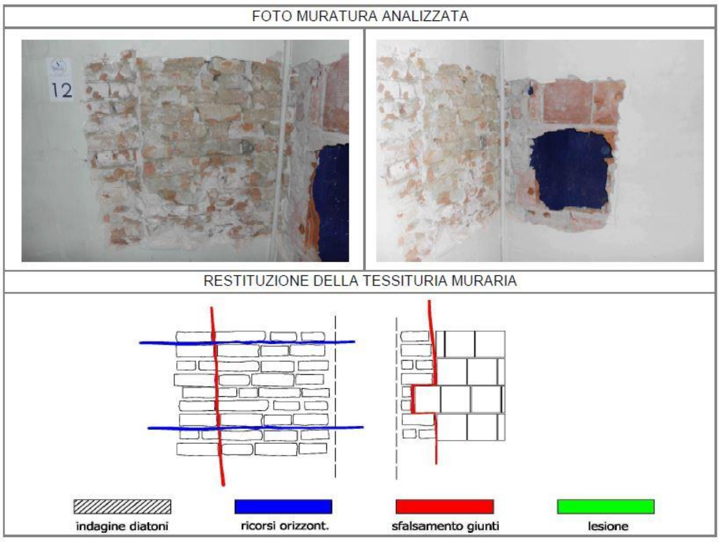


Figure 163. Investigation 12

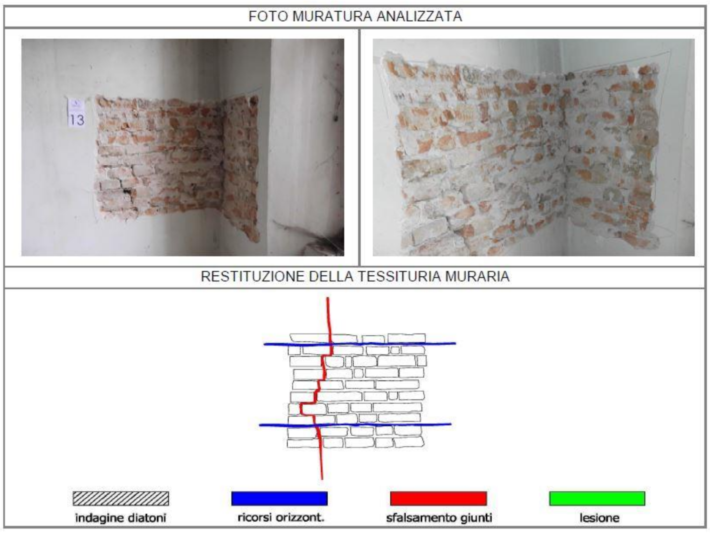


Figure 164. Investigation 13

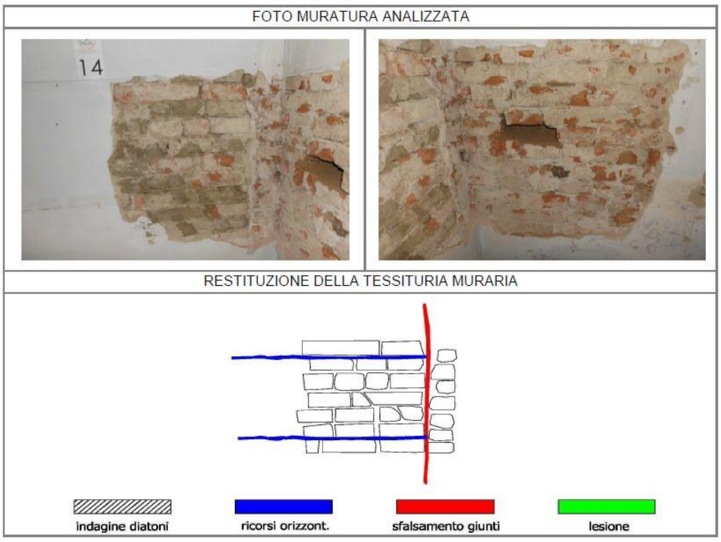


Figure 165. Investigation 14

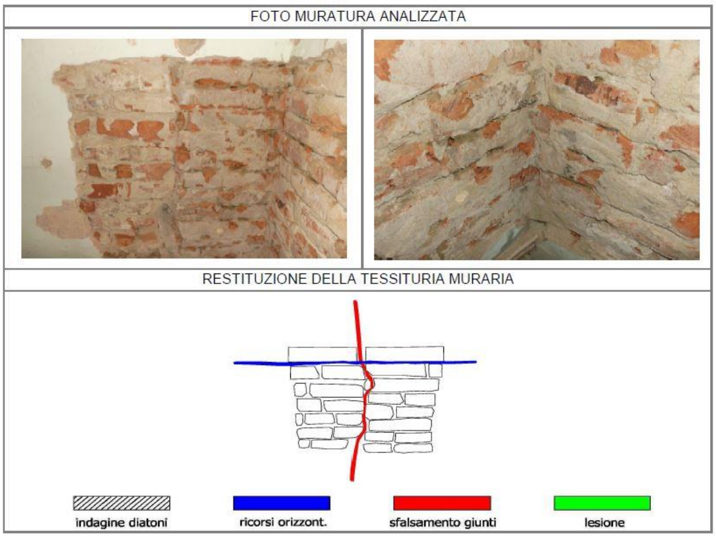


Figure 166. Investigation 15

In case of **aleatory uncertainties**, the objectives of the detailed investigations were to confirm/update the plausible mean value to be adopted in the final assessment.

For the group 1, I chose to effect the partially destructive test with double flat jacks, because the mechanical properties of masonry revealed a recurring parameter associated with a high sensitivity class and I wanted to achieve an high level of knowledge.

In this case, it can be reached through techniques of investigation that furnish some direct mechanical parameters but that, in the meantime, are not excessively invasive, like, for instance, the double flat jacks.

The masonry that characterizes the building is in fact very homogeneous in all of its areas and for bricks and mortar many studies and data of reference also exist in literature useful to support the evidences of a limited number of tests.

For group 5 (Constitutive laws of masonry panels) I have chosen to not perform any test because destructive tests would be necessary to assume more detailed information.

A reliable characterization of the behavior of masonry panels is generally achievable by destructive experimental tests (e.g., compression test, diagonal compression test) that allow direct characterization.

For groups 2 (Intermediate diaphragms stiffness), 3 (Roof diaphragms stiffness) and 6 (Vaults thickness), even if in the presence of medium or low sensitivity, I decided to opt for a high level of knowledge because it is less onerous, in terms of costs and invasiveness.

For group 4 (Staircases stiffness) I chose not to perform any test.

In cases of **epistemic uncertainty,** at the end of the investigations I chose the most reliable of the alternatives considered: the “flange effect”, associated with the quality of the connections, was considered to be effective.

This is a very important result because guaranteeing the connection is equivalent to guaranteeing an increased contribution to the resistance for the global system.

In this case, evidence of real damage, that showed no cracks between interior and exterior walls, supported the adoption of the good quality connection for the final assessment.

Indeed, the information obtained by Operational modal analysis(OMA) supported the adoption of good quality connection.

4.10. Seismic safety evaluation

For every parameter, the difference between the level associated with sensitivity class (SCk) and the knowledge level (LCk) results in a value, from 0 to 2 (when the difference is negative, this is set equal to 0), which quantifies the incomplete residual knowledge at the end of the investigations.

It is suitable as RU (Residual uncertainty) and calculated therefore as:

RESIDUAL UNCERTAINTY	
RUk=SCk-LCk	RU=max(RUk) (k=1,...N)

Numerical values 1,2,3 are attributed to the varying SCk and LCk, in correspondence of the level reached: high(H), medium(M), low(L).

The parameters that introduce RUk=0 are those for which knowledge is complete (LCH) or, if incomplete, do not have any meaningful influence on the answer.

In the case in which RUk assumes the value 1 or 2, the performed investigations have not been enough to reduce completely the sensibility to that parameter.

Consequently, there remains, despite the investigations, one residual uncertainty regarding the safety evaluation.

ANALYSIS N.2 (+X/MODE1/NO ECC.)		SLO		
RESIDUAL UNCERTAINTY				
Gk	SC	**LCk**	**RUk**	
G1 - mechanical parameters	H(3)	H(3)	0	
G2 - intermediate diaphragms stiffness	L(1)	H(3)	0	
G3 - roof diaphragms stiffness	L(1)	H(3)	0	
G4 - staircases stiffness	L(1)	L(1)	0	
G5 - constitutive laws of masonry panels	H(3)	L(1)	2	CFA
G6 - vaults thickness	M(2)	H(3)	0	

ANALYSIS N.2 (+X/MODE1/NO ECC.)		SLD		
RESIDUAL UNCERTAINTY				
Gk	SC	**LCk**	**RUk**	
G1 - mechanical parameters	H(3)	H(3)	0	
G2 - intermediate diaphragms stiffness	L(1)	H(3)	0	
G3 - roof diaphragms stiffness	L(1)	H(3)	0	
G4 - staircases stiffness	L(1)	L(1)	0	
G5 - constitutive laws of masonry panels	H(3)	L(1)	2	CFA
G6 - vaults thickness	L(1)	H(3)	0	

ANALYSIS N.2 (+X/MODE1/NO ECC.)		SLV		
RESIDUAL UNCERTAINTY				
Gk	SC	**LCk**	**RUk**	
G1 - mechanical parameters	H(3)	H(3)	0	
G2 - intermediate diaphragms stiffness	L(1)	H(3)	0	
G3 - roof diaphragms stiffness	L(1)	H(3)	0	
G4 - staircases stiffness	L(1)	L(1)	0	
G5 - constitutive laws of masonry panels	H(3)	L(1)	2	CFA
G6 - vaults thickness	L(1)	H(3)	0	

ANALYSIS N.19 (+Y/MODE1/ECC. 159,6)		SLO		
RESIDUAL UNCERTAINTY				
Gk	SC	**LCk**	**RUk**	
G1 - mechanical parameters	H(3)	H(3)	0	
G2 - intermediate diaphragms stiffness	H(3)	H(3)	0	
G3 - roof diaphragms stiffness	L(1)	H(3)	0	
G4 - staircases stiffness	L(1)	L(1)	0	
G5 - constitutive laws of masonry panels	H(3)	L(1)	2	CFA
G6 - vaults thickness	L(1)	H(3)	0	

ANALYSIS N.19 (+Y/MODE1/ECC. 159,6)		SLD		
RESIDUAL UNCERTAINTY				
Gk	SC	**LCk**	**RUk**	
G1 - mechanical parameters	H(3)	H(3)	0	
G2 - intermediate diaphragms stiffness	H(3)	H(3)	0	
G3 - roof diaphragms stiffness	L(1)	H(3)	0	
G4 - staircases stiffness	L(1)	L(1)	0	
G5 - constitutive laws of masonry panels	H(3)	L(1)	2	CFA
G6 - vaults thickness	L(1)	H(3)	0	

ANALYSIS N.19 (+Y/MODE1/ECC. 159,6)		SLV		
RESIDUAL UNCERTAINTY				
Gk	SC	LCk	RUk	
G1 - mechanical parameters	H(3)	H(3)	0	
G2 - intermediate diaphragms stiffness	L(1)	H(3)	0	
G3 - roof diaphragms stiffness	L(1)	H(3)	0	
G4 - staircases stiffness	L(1)	L(1)	0	
G5 - constitutive laws of masonry panels	H(3)	L(1)	2	CFA
G6 - vaults thickness	L(1)	H(3)	0	

Figure 167. Calculation of RU (Residual uncertainty)

As can be seen from the above charts, at the end of the investigations, **“Group 5” (constitutive laws of masonry panels) was the most sensitive parameter**.

In all cases it is necessary to apply a Confidence Factor CF in the verification, which must be applied to the parameter k*, selected among the parameters for which RUk is maximum and, possibly, with SCk=3 (SCH).

This is the most important step of the sensitivity analysis because:

- Knowledge Level is tuned on each parameter or constructive detail in connection with its influence on the seismic behavior rather than its assignment on a global scale;
- Knowledge Levels are differentiated depending on the amount and quality of collected information.

The verification is performed through the analysis with a model in which the central values Gk mean is attributed to all the parameters G1-G2-G3-G4-G6, while, to the parameter G5, which introduces the greatest residual uncertainty(k*) is assigned the following value, modified through CF:

$$X_{K*} = CF \quad X_{K*,mean} \tag{7}$$

$$CF = \frac{1}{3}\left[2 - RU* + (RU*+1)^{X_{K*,\min} / X_{K*,mean}}\right] \tag{8}$$

where:

$Xk*_{,min}$ is the extreme value of the interval of parameter k* that produces the smallest value of the PGA_{SL} (according to the cases of the superior extreme $Xk*_{,up}$ or inferior $Xk*_{,low}$).

FINAL MODEL VALUES				
Gk		**Gk, preliminary model**	**CF_A**	**Gk, final model**
G5 –constitutive laws of masonry panels	Shear-force drift(%)	0,004	0,769	0,003076
	Flexion-force drift(%)	0,006	0,666	0,003996

PRELIMINARY MODEL VALUES				
Gk		**Gk,low**	**Gk,up**	**Gk,mean**
G5 –constitutive laws of masonry panels	Shear-force drift(%)	0,0025	0,004	0,00325
	Flexion-force drift(%)	0,004	0,008	0,006

Figure 168. Modified final values through Confidence Factors "CF".

The CF adopted for the parameters that most affect the structural response (G5), derived using equation (8), allowed me to update the values of shear-force drift(%) and flexion-force drift(%) to be used for the final model.

The final seismic safety evaluation is assessed adopting a final model with updated parameters and performing pushover analyses.

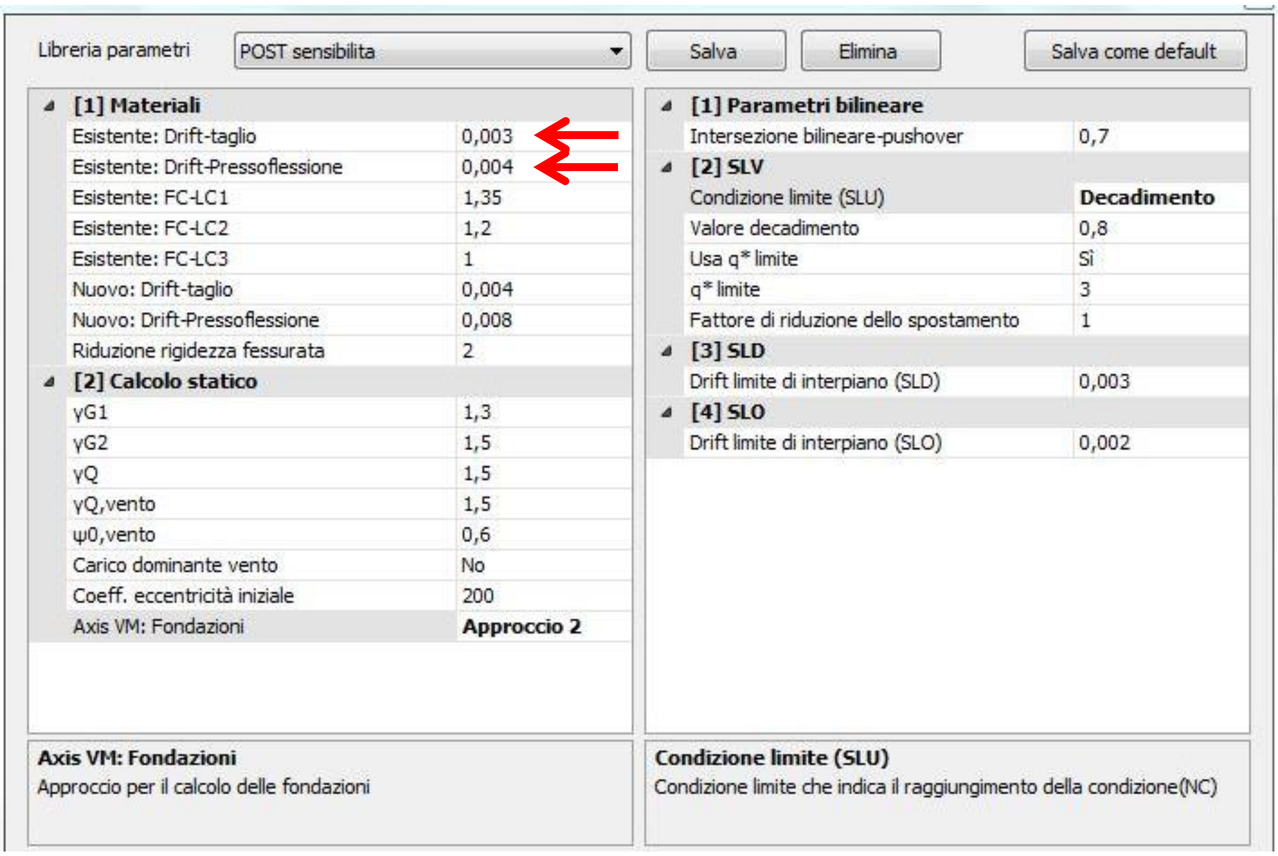


Figure 169. Mechanical parameters updated

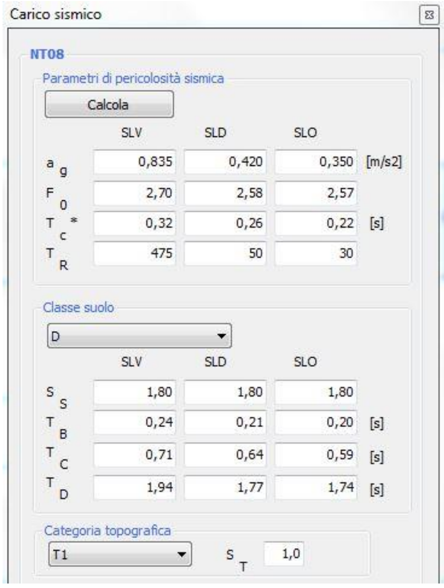


Figure 170. Seismic parameters definition

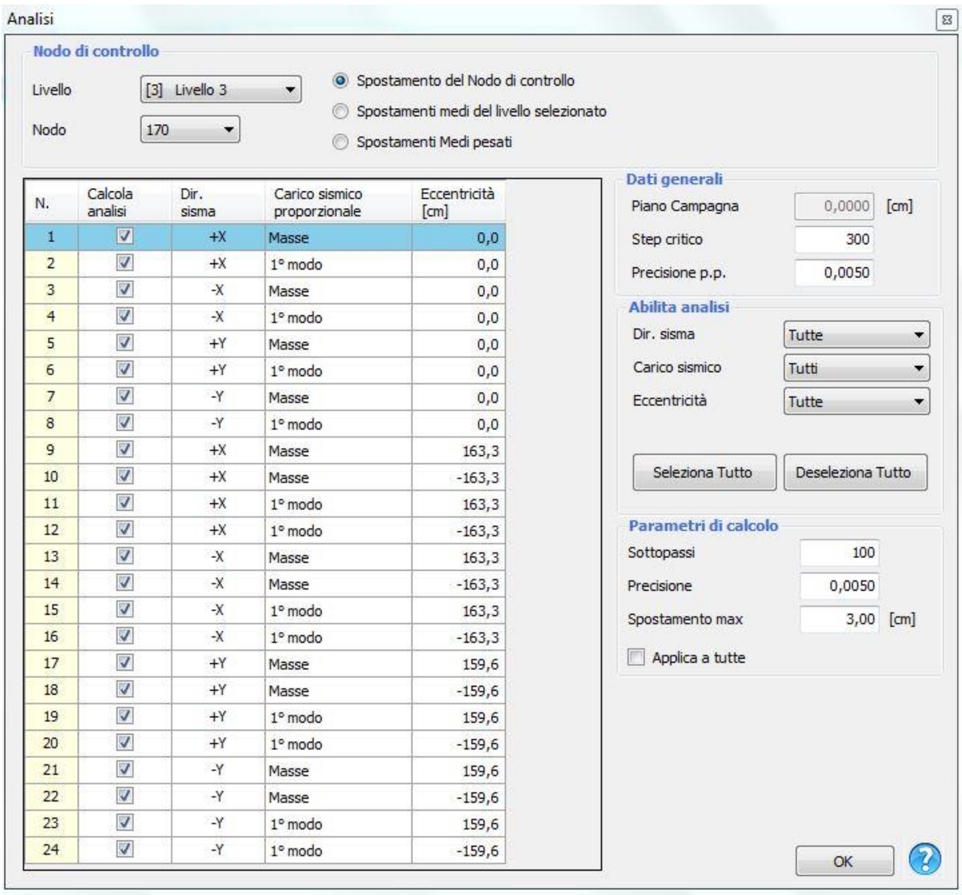


Figure 171. Calculation parameters definition

Verifica analisi

N.	Inserisci in relazione	Dir. sisma	Carico sismico proporzionale	Eccentricità [cm]	Dmax SLV [cm]	Du SLV [cm]	q* SLV	Dmax SLD [cm]	Du SLD [cm]	Dmax SLO [cm]	Do SLO [cm]	α SLV	α SLD	α SLO
1	☑	+X	Masse	0,00	1,36	0,57	1,77	0,41	0,57	0,34	0,57	0,609	1,280	1,556
2	☑	+X	1° modo	0,00	1,39	0,47	2,88	0,48	0,46	0,35	0,46	0,459	0,977	1,210
3	☑	-X	Masse	0,00	0,99	1,61	1,34	0,37	0,82	0,30	0,82	1,367	1,911	2,349
4	☑	-X	1° modo	0,00	1,70	1,25	2,30	0,53	0,56	0,41	0,56	0,800	1,026	1,252
5	☑	+Y	Masse	0,00	0,34	0,43	0,96	0,16	0,43	0,13	0,43	1,166	2,460	3,002
6	☑	+Y	1° modo	0,00	0,41	0,70	1,13	0,18	0,45	0,15	0,45	1,425	2,255	2,769
7	☑	-Y	Masse	0,00	0,35	0,56	0,70	0,17	0,56	0,14	0,52	1,533	3,219	3,708
8	☑	-Y	1° modo	0,00	0,38	0,60	0,82	0,18	0,60	0,15	0,60	1,425	3,026	3,711
9	☑	+X	Masse	163,25	1,43	1,12	1,70	0,45	1,08	0,38	1,08	0,855	1,811	2,258
10	☑	+X	Masse	-163,25	1,41	0,55	1,86	0,42	0,55	0,35	0,55	0,578	1,214	1,474
11	☑	+X	1° modo	163,25	1,62	1,12	1,44	0,78	1,11	0,65	1,11	0,693	1,425	1,716
12	☑	+X	1° modo	-163,25	1,64	0,60	2,32	0,51	0,60	0,39	0,60	0,514	1,094	1,344
13	☑	-X	Masse	163,25	1,33	1,45	1,52	0,45	0,98	0,37	0,98	1,056	1,788	2,209
14	☑	-X	Masse	-163,25	1,23	1,54	1,54	0,44	1,17	0,36	1,17	1,165	2,096	2,622
15	☑	-X	1° modo	163,25	1,68	1,28	2,24	0,52	0,57	0,41	0,57	0,821	1,055	1,288
16	☑	-X	1° modo	-163,25	1,73	1,23	2,43	0,56	0,53	0,41	0,53	0,775	0,973	1,187
17	☑	+Y	Masse	159,59	0,37	1,35	1,34	0,14	0,28	0,12	0,28	2,245	1,781	2,174
18	☑	+Y	Masse	-159,59	0,37	0,86	0,81	0,18	0,86	0,15	0,63	1,848	4,018	3,922
19	☑	+Y	1° modo	159,59	0,38	0,30	1,15	0,17	0,30	0,14	0,30	0,871	1,811	2,182
20	☑	+Y	1° modo	-159,59	0,42	1,46	0,97	0,20	1,18	0,17	0,51	2,509	4,683	2,919
21	☑	-Y	Masse	159,59	0,38	0,80	1,46	0,14	0,40	0,12	0,40	1,722	2,240	2,805
22	☑	-Y	Masse	-159,59	0,38	0,88	0,73	0,18	0,72	0,15	0,58	1,909	3,552	3,733
23	☑	-Y	1° modo	159,59	0,35	0,43	0,83	0,17	0,43	0,14	0,43	1,207	2,511	3,025
24	☑	-Y	1° modo	-159,59	0,43	1,41	0,83	0,21	0,64	0,17	0,49	2,485	2,920	2,876

Legenda colori

Verificato — Non verificato — Non converge a p.p. — Analisi più gravosa

Figure 172. Seismic computations results (final model)

The check compares the two displacements offered by the structure and required by the code.

If the first is greater than the second, the structure satisfies the check.

This window shows the results of the seismic computations performed on the model and summarizes the check parameters according to each norm, indicating whether the results were satisfactory or not.

The first columns describe the type of analysis, the last shows the vulnerability indexes for each of the three limit states.

The background color, green or red, distinguishes between the analyses that are exceeded and those that are not.

The yellow color shows the two analyzes that have the lowest vulnerability indexes (more significant for the purposes of calculation).

The analyses that have minimum "Alfa" values are more restrictive, so the results window shows the two analyses with the minimum "Alfa SLV" (one for the X direction and one for the Y direction).

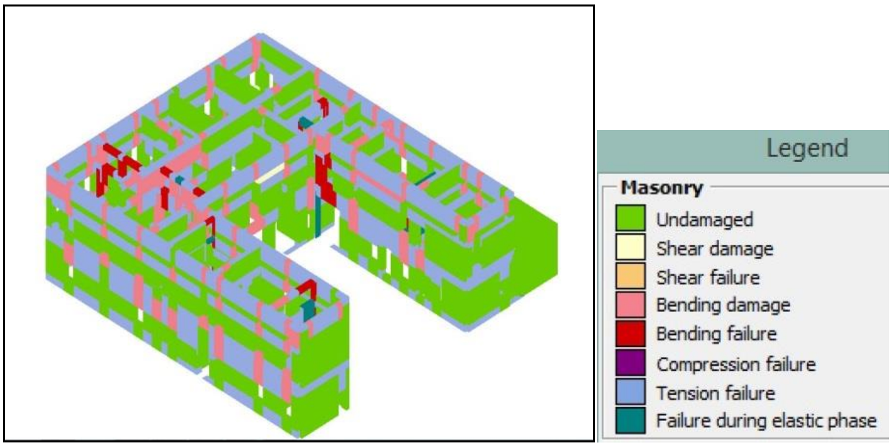


Figure 173. Analysis N.2 –“X” direction

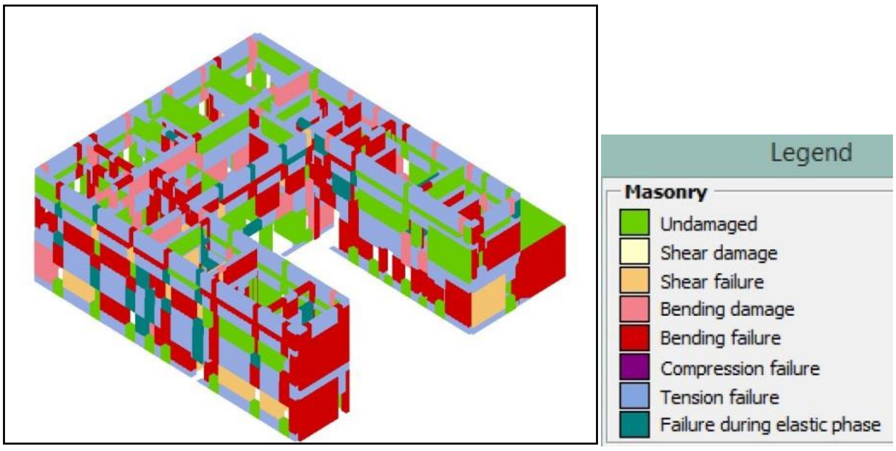
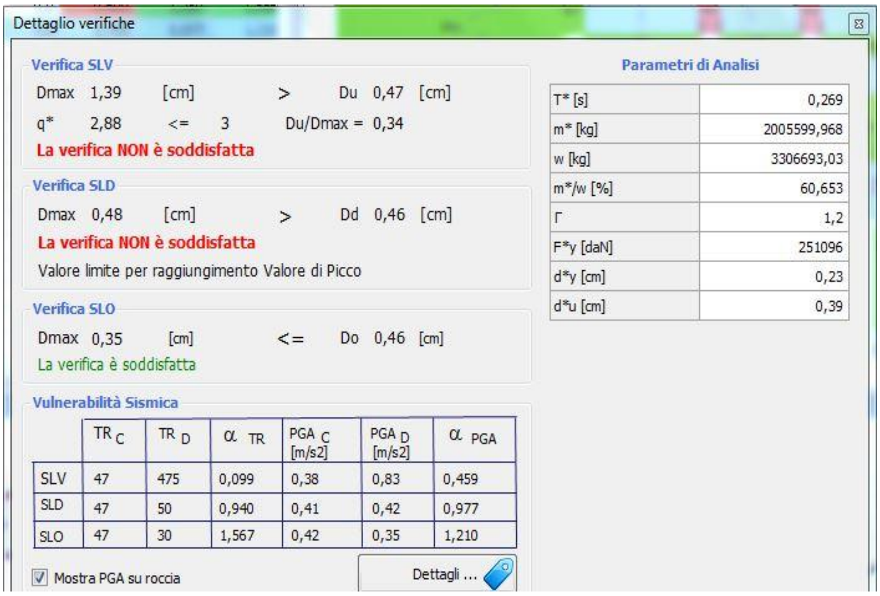


Figure 174. Analysis N.19 – “Y” direction


Dettaglio verifiche

Verifica SLV

Dmax 1,39 [cm] > Du 0,47 [cm]

q* 2,88 <= 3 Du/Dmax = 0,34

La verifica NON è soddisfatta

Verifica SLD

Dmax 0,48 [cm] > Dd 0,46 [cm]

La verifica NON è soddisfatta

Valore limite per raggiungimento Valore di Picco

Verifica SLO

Dmax 0,35 [cm] <= Do 0,46 [cm]

La verifica è soddisfatta

Vulnerabilità Sismica

	TR C	TR D	α TR	PGA C [m/s2]	PGA D [m/s2]	α PGA
SLV	47	475	0,099	0,38	0,83	0,459
SLD	47	50	0,940	0,41	0,42	0,977
SLO	47	30	1,567	0,42	0,35	1,210

Mostra PGA su roccia

Dettagli ...

Parametri di Analisi

T* [s]	0,269
m* [kg]	2005599,968
w [kg]	3306693,03
m*/w [%]	60,653
Γ	1,2
F*y [daN]	251096
d*y [cm]	0,23
d*u [cm]	0,39

Figure 175. Analysis N.2 – “X” direction results details table

Dettaglio verifiche

Verifica SLV

Dmax 0,38 [cm] > Du 0,30 [cm]

q* 1,15 <= 3 Du/Dmax = 0,79

La verifica NON è soddisfatta

Verifica SLD

Dmax 0,17 [cm] <= Dd 0,30 [cm]

La verifica è soddisfatta

Valore limite per raggiungimento Valore di Picco

Verifica SLO

Dmax 0,14 [cm] <= Do 0,30 [cm]

La verifica è soddisfatta

Parametri di Analisi

T* [s]	0,32
m* [kg]	3463214,345
w [kg]	3306693,03
m*/w [%]	104,733
Γ	0,37
F*y [daN]	1087477
d*y [cm]	0,81
d*u [cm]	0,81

Vulnerabilità Sismica

	TR_C	TR_D	α_{TR}	PGA_C [m/s2]	PGA_D [m/s2]	α_{PGA}
SLV	306	475	0,644	0,73	0,83	0,871
SLD	306	50	6,120	0,76	0,42	1,811
SLO	306	30	10,200	0,76	0,35	2,182

☑ Mostra PGA su roccia Dettagli ...

Figure 176. Analysis N.19 – “Y” direction results details table

These represents two summary windows that display the details of the analyses and required checks.

Vulnerabilità Sismica

						TR_C			TR=cost	
	TR_C	TR_D	α_{TR}	PGA_D [m/s2]	$PGA_C(TR)$ [m/s2]	$F_0(TR)$	$T^*_C(TR)$	$\alpha_{PGA}(TR)$	PGA_C [m/s2]	α_{PGA}
SLV	47	475	0,099	0,83	0,41	2,58	0,25	0,492	0,38	0,459
SLD	47	50	0,940	0,42	0,41	2,58	0,25	0,979	0,41	0,977
SLO	47	30	1,567	0,35	0,41	2,58	0,25	1,174	0,42	1,210

Figure 177. Analysis N.2 – “X” direction seismic vulnerability

Vulnerabilità Sismica

						TR_C			TR=cost	
	TR_C	TR_D	α_{TR}	PGA_D [m/s2]	$PGA_C(TR)$ [m/s2]	$F_0(TR)$	$T^*_C(TR)$	$\alpha_{PGA}(TR)$	PGA_C [m/s2]	α_{PGA}
SLV	306	475	0,644	0,83	0,74	2,66	0,31	0,883	0,73	0,871
SLD	306	50	6,120	0,42	0,74	2,66	0,31	1,756	0,76	1,811
SLO	306	30	10,200	0,35	0,74	2,66	0,31	2,108	0,76	2,182

Figure 178. Analysis N.19 – “Y” direction seismic vulnerability

The "Seismic Vulnerability" evaluation table shows the parameters derived from the homonyms reports for each of the limit states:

$\alpha_{PGA}=PGA_C/PGA_D$

$\alpha_{TR}=TR_C/TR_D$

- **PGA$_C$**: Limit capacity acceleration for each limit state (independent from the seismic spectrum).
- **PGA$_D$**: Spectral acceleration for each of the limit states (depends on the seismic spectrum).
- **TR$_C$**: Return period of the limit capacity seismic action for each of the limit states.
- **TR$_D$**: Spectral return period for each of the limit states.

Identifying the parameters that most affect the structural response allowing to the optimization of the investigation plan and the calibration of the value of confidence factor, the average value of "αPGA(SLV)" in "X" direction increased by 14%.

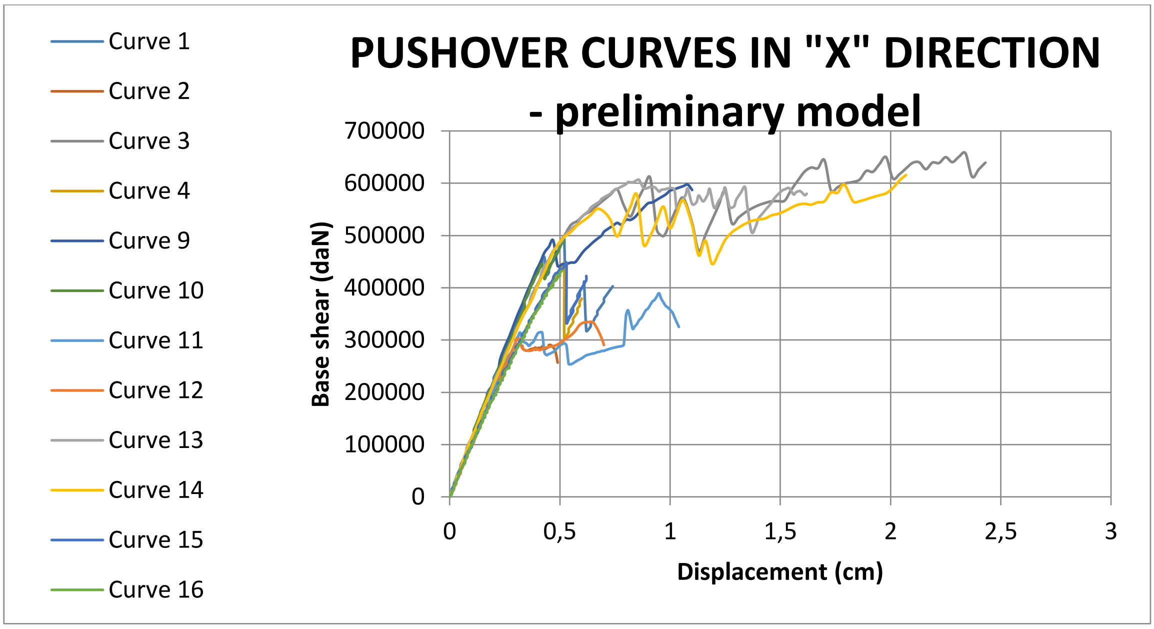


Figure 179. Preliminary model - direction "X" analyses

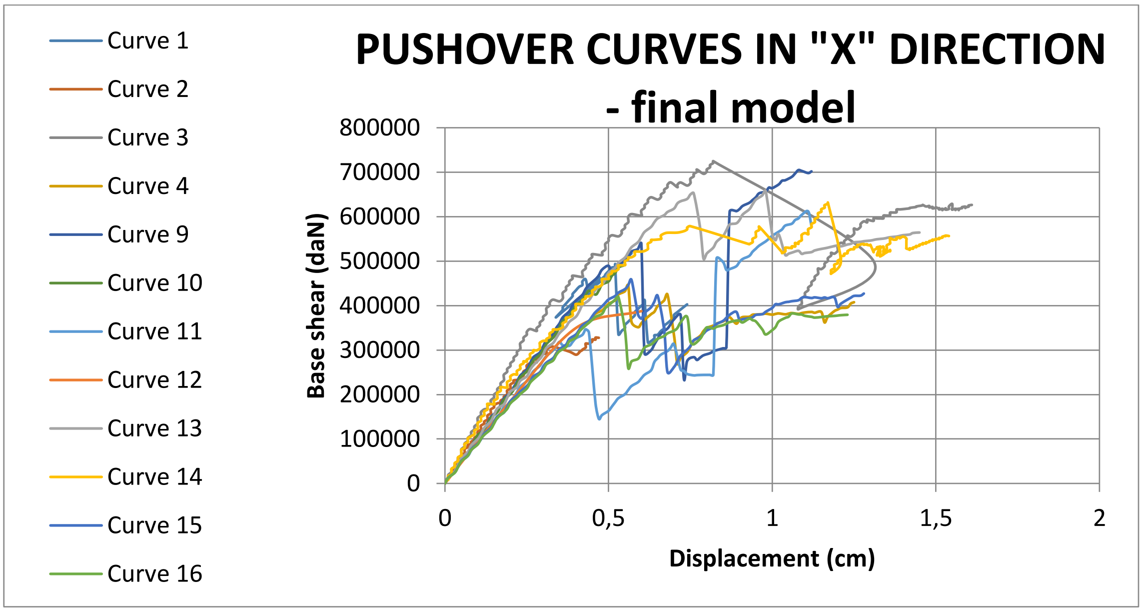


Figure 180. Final model - direction "X" analyses

Identifying the parameters that most affect the structural response allow the optimization of the investigation plan and the calibration of the value of the confidence factor, the average value of "αPGA(SLV)" in "Y" direction increased by 35%.

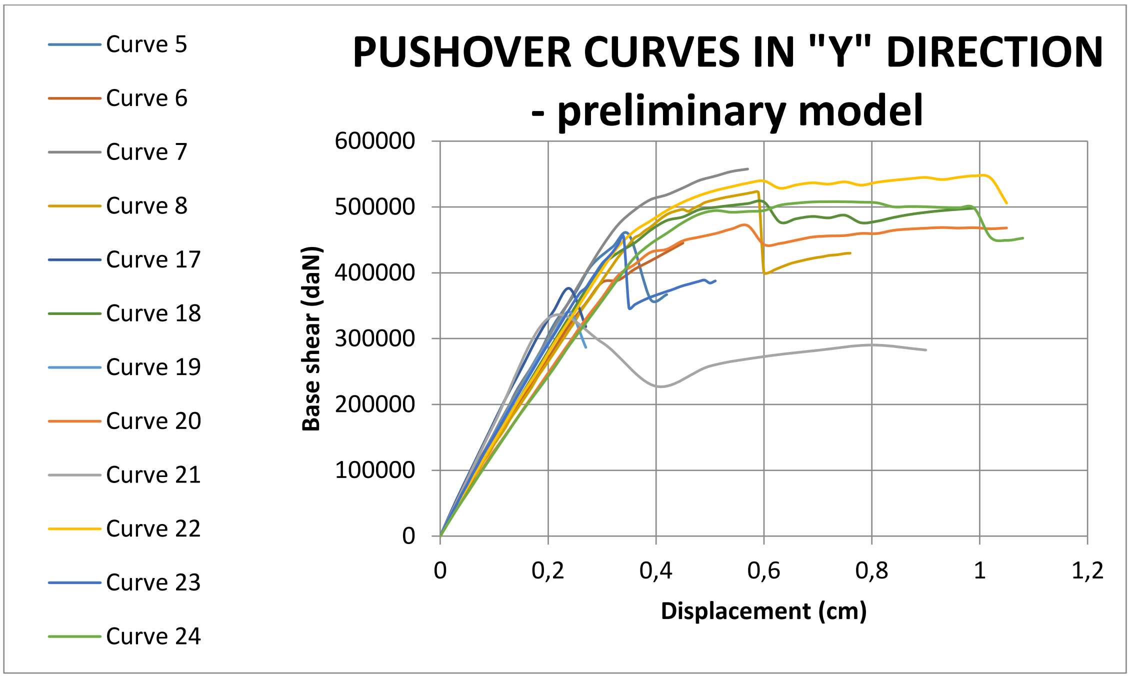


Figure 181. Preliminary model - direction "Y" analyses

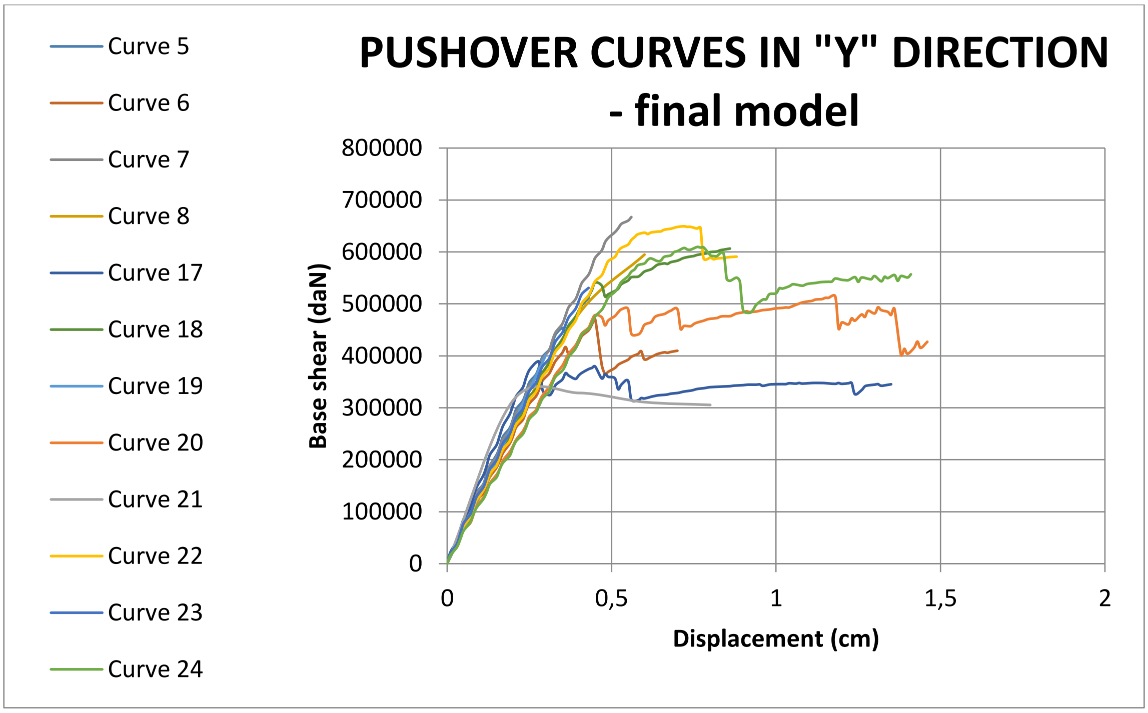


Figure 182. Final model - direction "Y" analyses

Incomplete knowledge in the seismic assessment of existing masonry structures is usually addressed in standards through a deterministic procedure based on the use of confidence factors for the whole building.

The results obtained show that the probabilistic procedure through the coded use of the sensitivity analysis brought the following advantages:

- the required quantity of tests was calibrated by determining the structure reaction to changes in the resistance parameters of the materials, thus avoiding performing tests in insignificant points and instead extending the greatest impact knowledge areas;
- the optimization of investigations allowed not only the reduction of the impact of *in-situ* tests, but also the reduction of costs;
- the results obtained by not applying a single confidence factor to whole building were less penalizing.
- the reduction of uncertainties may result in a higher risk index and therefore in a simpler and cheaper structural intervention for the seismic retrofitting of the building.

4.11. Structural damage identification: FEM model and real damage experience

In this section, the seismic response obtained by the FEM model is compared with the damage experienced following the seismic events of 2012 in Emilia.

Specifically, the walls that were most damaged by the earthquake were analyzed to assess the compliance with the reality of the calculation program and the different results of the nonlinear static analyses are illustrated above.

The seismic response of the most damaged walls was obtained through the two analyses that have the lowest vulnerability indices considering the X or the Y direction.

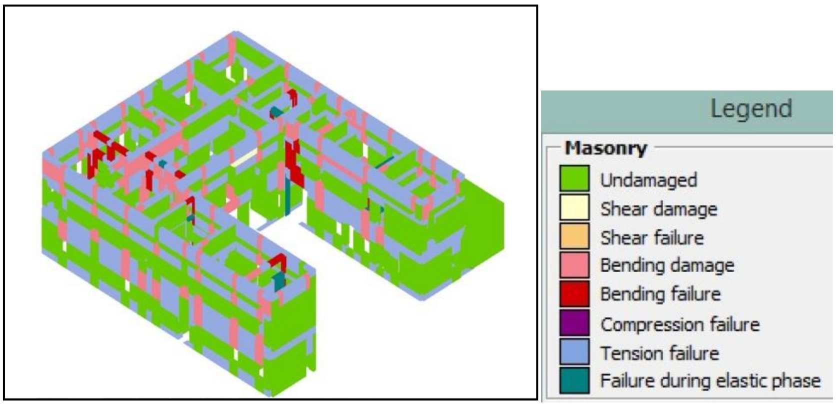


Figure 173*. Analysis N.2 –“X” direction

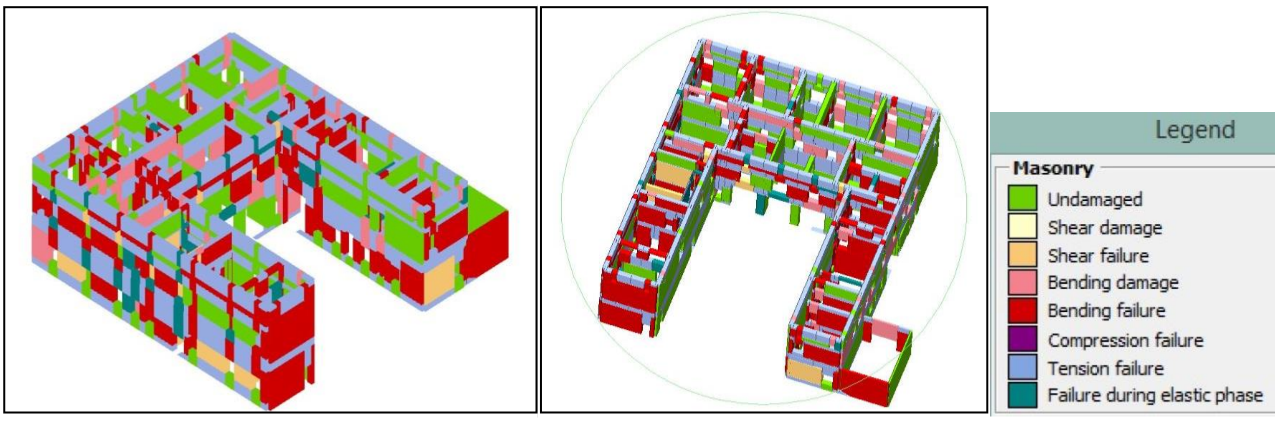


Figure 174*. Analysis N.19 –“Y” direction

Figure 183 shows the identification of wall 3 and the damage pattern of the masonry panel is illustrated in the following figures.

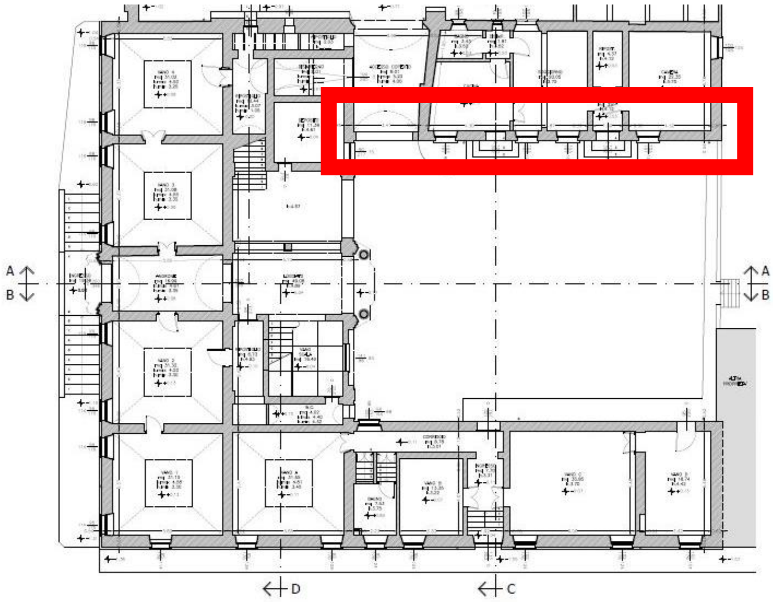


Figure 183. Wall identification: wall 3

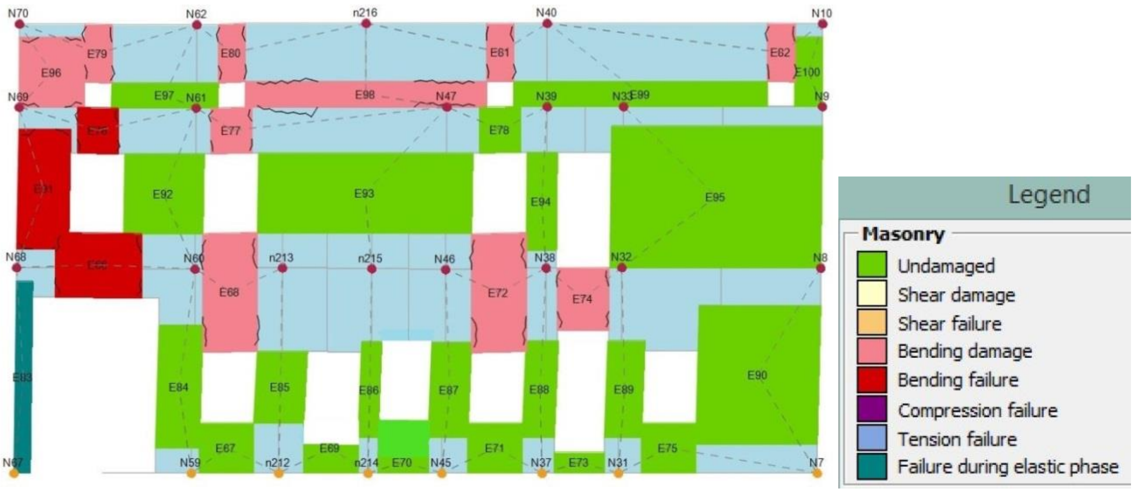


Figure 184. X direction_damage pattern with related legend of wall 3

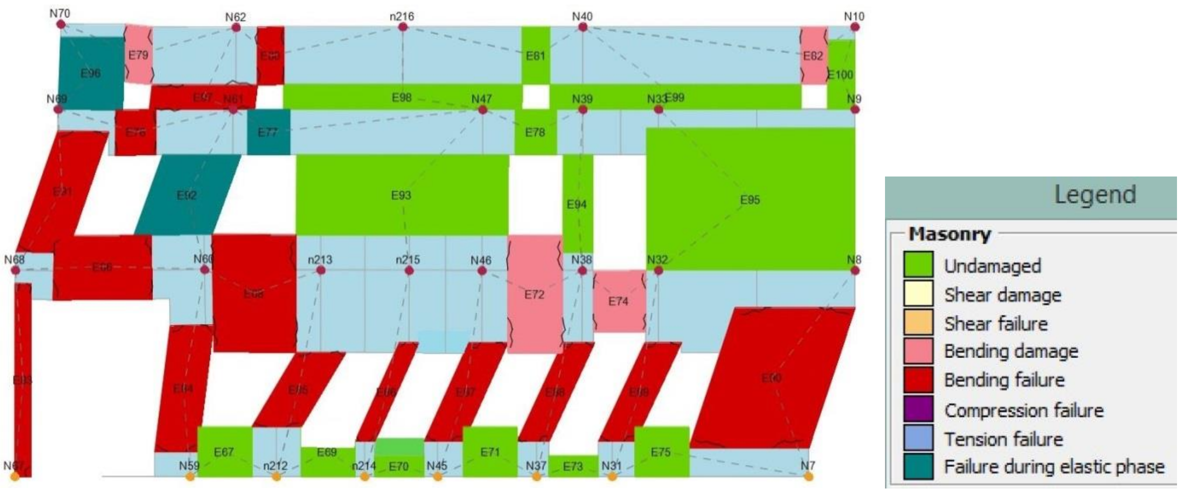


Figure 185. Y direction_damage pattern with related legend of wall 3

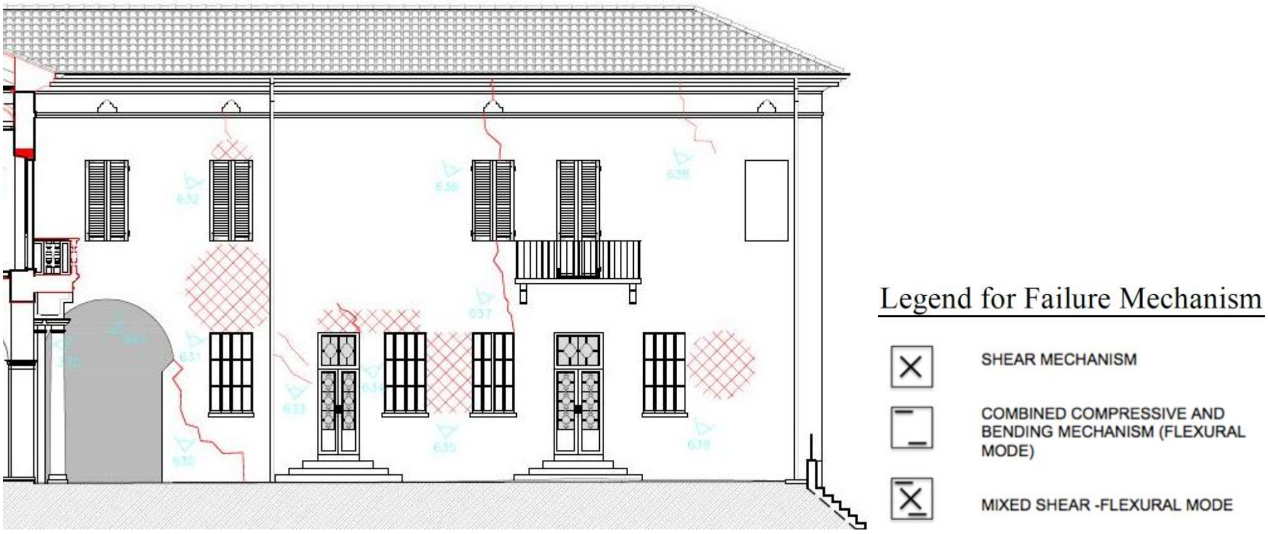


Figure 186. Seismic events 2012 in Emilia_ damage suffered by wall 3

Figure 187-a. Seismic events 2012 in Emilia_ damage suffered by wall 3

Figure 187-b/c. Seismic events 2012 in Emilia_ damage suffered by wall 3

The most significant analyses for calculation in the X and Y direction show that wall 3 is subjected to a significant local deformation, especially to the wall-to-wall connection.

As can be observed in Figures 187-a/b/c, the facade is not able to absorb the tensions to which it is subjected due to the structural discontinuity, unlike the other walls less subjected to transformations.

The damage in that area is facilitated by the presence of a large opening that interrupts the structural continuity right near the wall crossing thus making wall 3 vulnerable.

Regarding the damage, the recurring collapse mechanism in both directions is the combination of compression and bending (flexural mode), particularly localized at the ground floor.

The wall damage mechanism resulting from the FEM model can be assimilated with that reported after the earthquake, although, to the right of the large opening on the ground floor, the breaking of the wall happened due to the effect of the shear mechanism.

Figure 188 shows the identification of wall 4 and the damage pattern of the masonry panel is illustrated in the following figures.

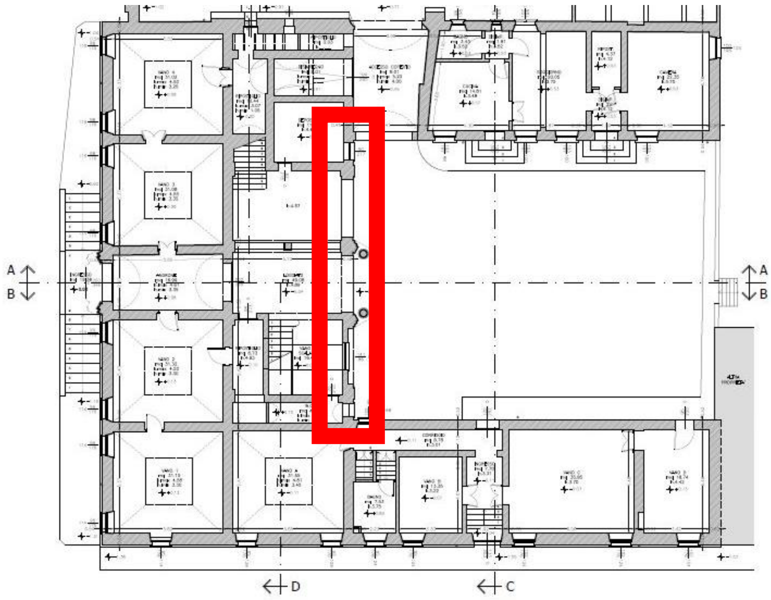


Figure 188. Wall identification: wall 4

Figure 189. X direction_damage pattern with related legend of wall 4

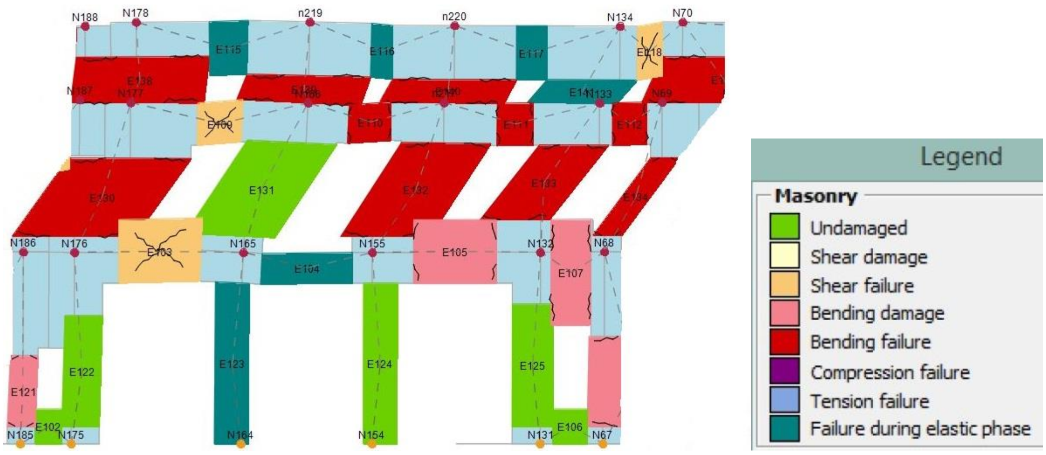


Figure 190. Y direction_damage pattern with related legend of wall 4

Figure 191. Seismic events 2012 in Emilia_ damage suffered by wall 4

Figure 192-a. Seismic events 2012 in Emilia_ damage suffered by wall 4

Figure 192-b-c. Seismic events 2012 in Emilia_ damage suffered by wall 4

The most significant analyses for calculation in the X and Y direction show that wall 4 is subjected to significant distortions especially in the Y direction.

As can be observed in Figures 192-a/b/c, the facade is not able to absorb the tensions to which it is subjected due to the structural discontinuity.

The damage in that area is facilitated by the presence of large openings that interrupt structural continuity and make wall 4 very vulnerable.

Regarding the damage, the recurring collapse mechanism in X direction is the bending damage of some masonry spandrels, while the recurring collapse mechanism in Y direction is the combination of compressing and bending (flexural mode), particularly localized at the first and second floor.

There are also some shear failures in some spandrels and other failures that occurred during the elastic phase, especially on the upper floors.

From the comparison between the FEM model and the damage experienced following the seismic events, it is possible to note that the mechanism of damage of the second floor corresponds to what really happened.

Figure 193 shows the identification of wall 4 and the damage pattern of the masonry panel is illustrated in the following figures.

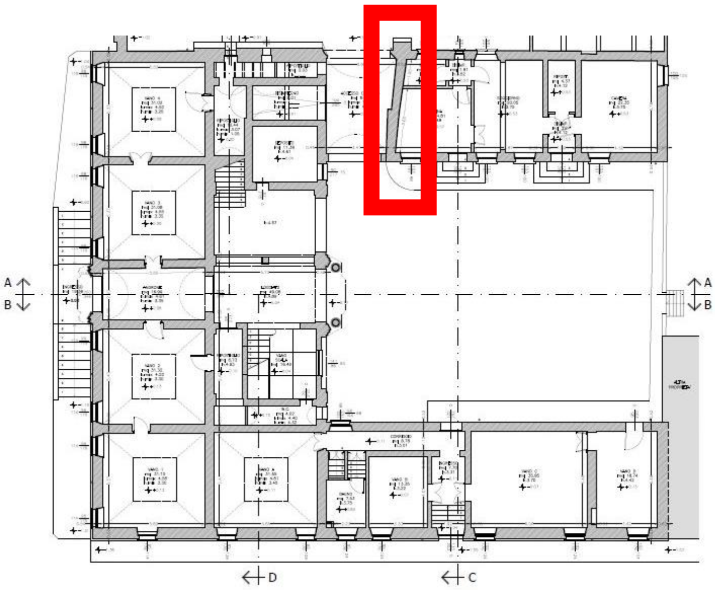


Figure 193. Wall identification: wall 14

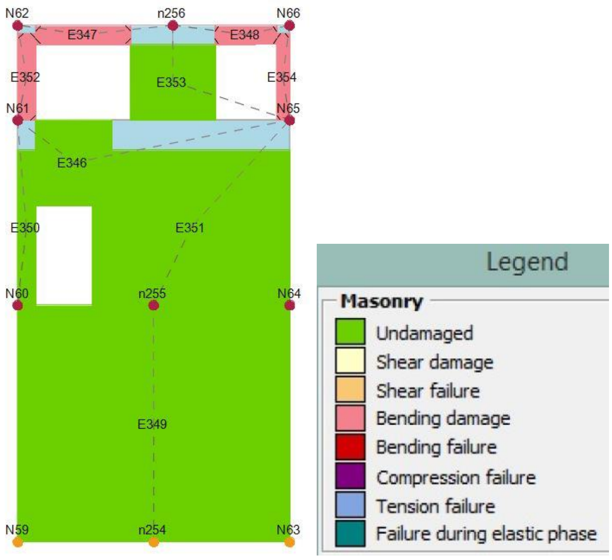


Figure 194. X direction_ damage pattern with related legend of wall 14

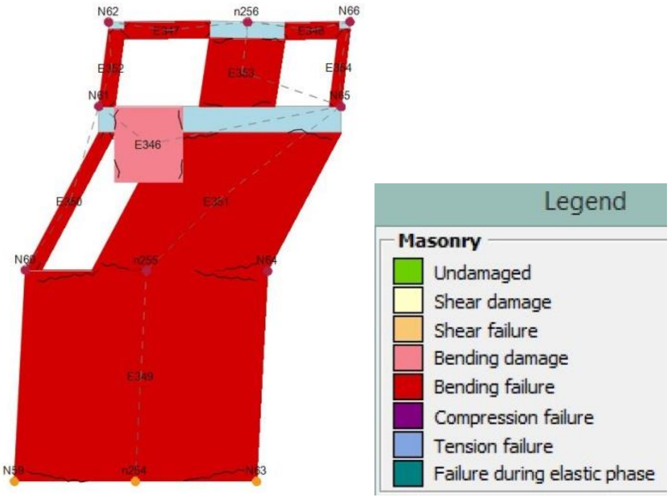


Figure 192. Y direction_ damage pattern with related legend of wall 14

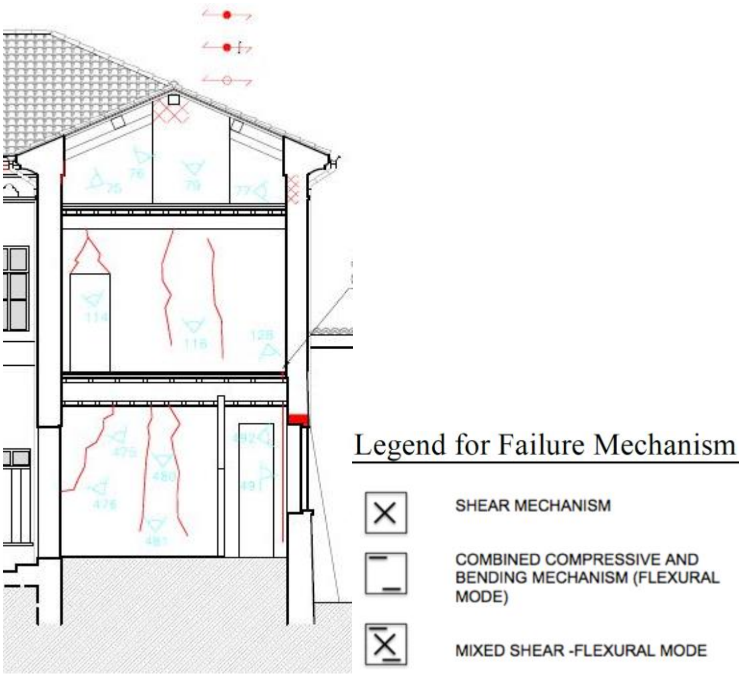


Figure 193. Seismic events 2012 in Emilia_ damage suffered by wall 14

Figures 194-195. Seismic events 2012 in Emilia_ damage suffered by wall 14 on the ground floor


Figures 196-197. Seismic events 2012 in Emilia_ damage suffered by wall 14 on the first floor


Figures 198-199. Seismic events 2012 in Emilia_ damage suffered by wall 14 on the second floor

As can be observed in Figures 194-195-196-197-198-199, the damage that has brought back the facade 14 has shown different critical situations as the presence of flues, recesses, infill openings and not continuous walls.

Regarding the damage, the recurring collapse mechanism in Y direction is the bending failure of almost all masonry piers and spandrels at every level, while in X direction the masonry turned out to be largely undamaged.

Comparing the seismic response obtained by the FEM model with the damage experienced following the seismic events, it is possible to note that the mechanism of damage of wall 14 is similar to the damage brought about by the wall due to the earthquake.

As was possible to see from the walls analyzed above, the adoption of the global model for the seismic assessment has produced rather realistic results, giving a correct interpretation of structural behavior even in a complex configuration like this.

In particular, the results of the analysis have shown that the seismic response is characterized by an interaction between the different bodies of the whole aggregate.

Current work has shown that a more realistic modelling of the structure leads to more realistic results and more realistic results can make it possible to achieve more targeted and effective structural intervention for the seismic retrofitting of the building with remarkable economic advantages.

5. CONCLUSIONS AND SUGGESTION FOR THE FURTHER WORK

5.1. Conclusions

My research aims to broaden the current database present in the literature regarding the seismic assessment of architectural heritage buildings and tries to provide a new key to reading the latest damage identification techniques.

Some results of considerable interest are introduced below, following the order in which they were dealt with in the previous chapters.

The first part of my research, in particular chapters 2 and 3, are devoted to acquiring some theoretical tools needed for the subsequent applications.

Indeed, in chapter 2 I analyzed scientific literature related to the methodological principles of the sensitivity analysis in the seismic assessment of existing masonry buildings, while, in chapter 3, I analyzed scientific literature on damage diagnosis on masonry buildings, monitoring systems, measurement techniques and analytical methods to process data acquired.

In particular, minor and non-destructive tests were analyzed in detail because in some historical buildings such as the one used as a case study, destructive tests cannot be performed.

Instead, the second part of my research (chapter 4) was devoted to put into practice the theoretical tools previously acquired, adding interesting innovative aspects such as a preliminary calibration of the model through a careful evaluation of the rigidity of the diaphragms and the use of Operational modal analysis for the validation of the obtained results (sections 4.5 ,4.6,4.7).

The thesis focuses on the seismic performance assessment of a complex monumental masonry building as Palazzo Boldi, seriously damaged by the earthquake “Emilia Romagna – 2012”.

After a brief description of the biography of the architect who designed it(section 4.2), section 4.3 was entirely dedicated to the preliminary knowledge of the building, very delicate and challenging phase.

The detailed knowledge phase (including a careful historical, architectural and technological analysis) and the experimental campaign allowed the definition of constructive and mechanical features of the monument.

Section 4.4 was entirely dedicated to structural modeling but it was a very challenging issue: firstly, due to the correct interpretation of the structural behavior, which influences the accuracy of the seismic safety assessment; then, because of the several critical issues posed by the modelling of such a complex building, which has to be able to guarantee a reliable assessment, considering all possible variables.

Despite these difficulties, an accurate definition of the numerical model was achieved, so as to quantify the most appropriate level of safety.

The results obtained from the modal analysis were fundamental for a better understanding of the structural behavior of the building, highlighting differences in terms of modal forms, periods and mass participation coefficients.

The starting modeling with the horizontal structures shaped as deformable bodies has determined an irregular behavior due to a poor distribution of the actions through the horizontal diaphragms and a poor global response of the building, proving to be completely inadequate.

The subsequent modeling with the horizontal structures shaped as rigid bodies determined a more regular behavior, promoting a better distribution of the actions through the horizontal diaphragms and a better global response of the building.

The application of OMA techniques in this crucial phase served to demonstrate that this tool is now indispensable for the correct modelling of a building, especially if as complex as the one analyzed.

The innovative choice to use Operational modal analysis(OMA) to calibrate the structural behavior of the building has proved to be of crucial importance: information, such as modal forms and periods of oscillation, was measured and the real experimental values obtained allowed me to calibrate the correct stiffness of the horizontal structure, greatly improving preliminary knowledge.

This aspect of modeling is particularly relevant for a building characterized by the presence of different types of horizontal structures such as the one examined, in which the adoption of infinite rigidity is, in any case, inadequate.

For this reason, the adoption of semi-rigid diaphragms obtained by using an intermediate stiffness, was found to be the most appropriate and realistic for subsequent application: the sensitivity analysis.

The importance of the result obtained also lies in the fact that modelling the horizontal structures of the case study with a different stiffness would have led to completely wrong results or, in any case, results that were far from reality.

Other original results were obtained from the application of the procedures reported in the CNR-DT 212/2013 to the updated model .

It is based on the sensitivity analysis that guides the choice of *in-situ* characterization tests to improve the knowledge level of the structure(sections 4.8,4.9).

At the end of the investigations, the group of aleatory uncertainties which most significantly affect the seismic performance of the building was identified and only the confidence factor was applied to it(section 4.10); in the case of epistemic uncertainty, the most reliable of the considered alternatives was chosen.

In the section 4.10 the final seismic safety evaluation is assessed adopting a final model with updated parameters.

The comparison between the level of safety achieved by the deterministic procedure and that obtained by the probabilistic procedure yielded very significant results.

Specifically, the results obtained show that the level of the safety obtained using the deterministic procedure is more precautionary than the probabilistic procedure.

This means that the deterministic approach represents an approach that is too simplified to fully interpret the behavior of masonry buildings, ending up by underestimating the capabilities of the materials.

The use of the probabilistic procedure has brought the following advantages:

- the required quantity of tests was calibrated by determining the structure reaction to changes in the resistance parameters of the materials, thus avoiding performing tests in insignificant points and extending the knowledge areas of greatest impact instead;

- the optimization of investigations not only allowed the reduction of the impact of *in-situ* tests, but also the reduction of the costs;
- the results obtain, not applying a single confidence factor to whole building, were less penalizing.
- the reduction of uncertainties may result in a higher risk index and therefore in a simpler and cheaper structural intervention for the seismic retrofitting of the building.

This work also demonstrated the importance of evaluating global parameters, in addition to the mechanical properties of structural materials, such as the building's modal parameters(frequencies, mode shapes and damping ratios) which can be crucial to reducing the uncertainties in the assessment of the vulnerability of the building.

It can help to improve the CNR probabilistic procedure by demonstrating that simultaneous use of tools such as modal analysis and Operational modal analysis(OMA) can be based on a more accurate and realistic preliminary model, further limiting the inevitable uncertainties of modelling.

The end of chapter 4 (section 4.11), compares FEM model results with the damage experienced following the seismic events It can be said that the adoption of a global model for seismic assessment has produced quite realistic results, providing a correct interpretation of structural behavior even in a complex configuration like this.

5.2. Suggestions for further work

The use of dynamic tests together with sensitivity analysis represents a higher level of approach than current legislation and provides a more solid theoretical basis for the revision of current European rules on the seismic evaluation of existing structures.

Moreover, the procedure used in this thesis can offer an efficient tool to understand the safety of complex monumental buildings.

Despite obtaining some promising results, further work is needed in order to improve and enhance the current scientific literature.

There are still questions to be answered in more concrete terms as: “What type of intervention is most effective? In which area should they be applied?”

The problem of the knowledge of a building is surely a central issue even when a structural intervention has to be designed for the seismic retrofitting of the building.

A better definition of numerical models is necessary to quantify the safety levels and support the design of proper and effective strengthening interventions.

Moreover, a better knowledge of the structure also makes seismic reinforcement operations less expensive and less intrusive.

Further work will be addressed to the examination of some theoretical and practical aspects, such as:

· the study of the incidence of the uncertainties through “improvement sensitivity”. It will help to optimize the intervention strategies for the seismic retrofitting of the existing building, improving the intervention planning and maximizing the return in terms of structural improvement, proving to be a powerful aid for designers by finding more effective reinforcing interventions for the structure.

· the development and implementation, thanks to research and engineering practice, of a complete software able to manage all analysis procedures (static/dynamic), and able to perform increasingly realistic results.

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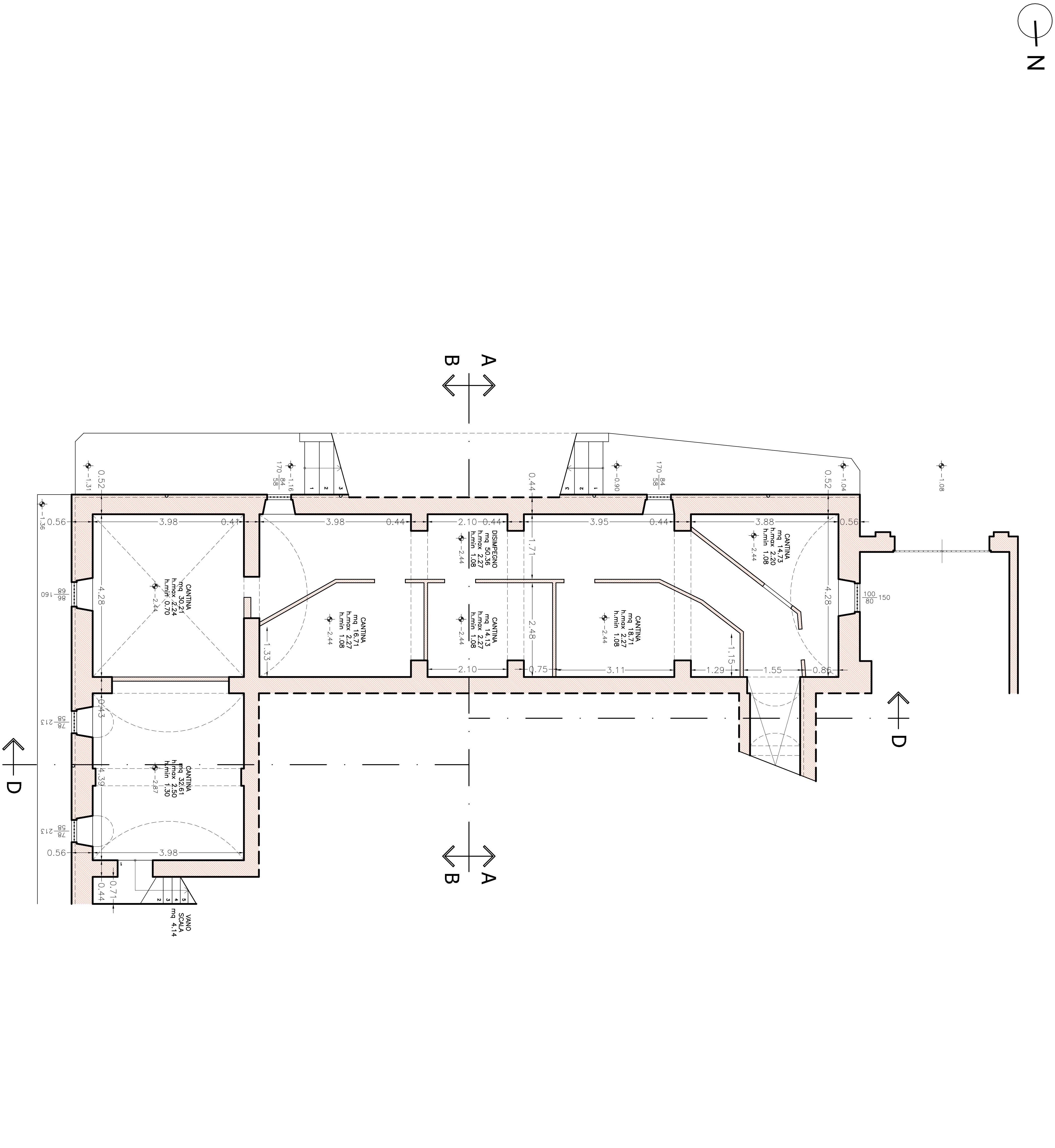
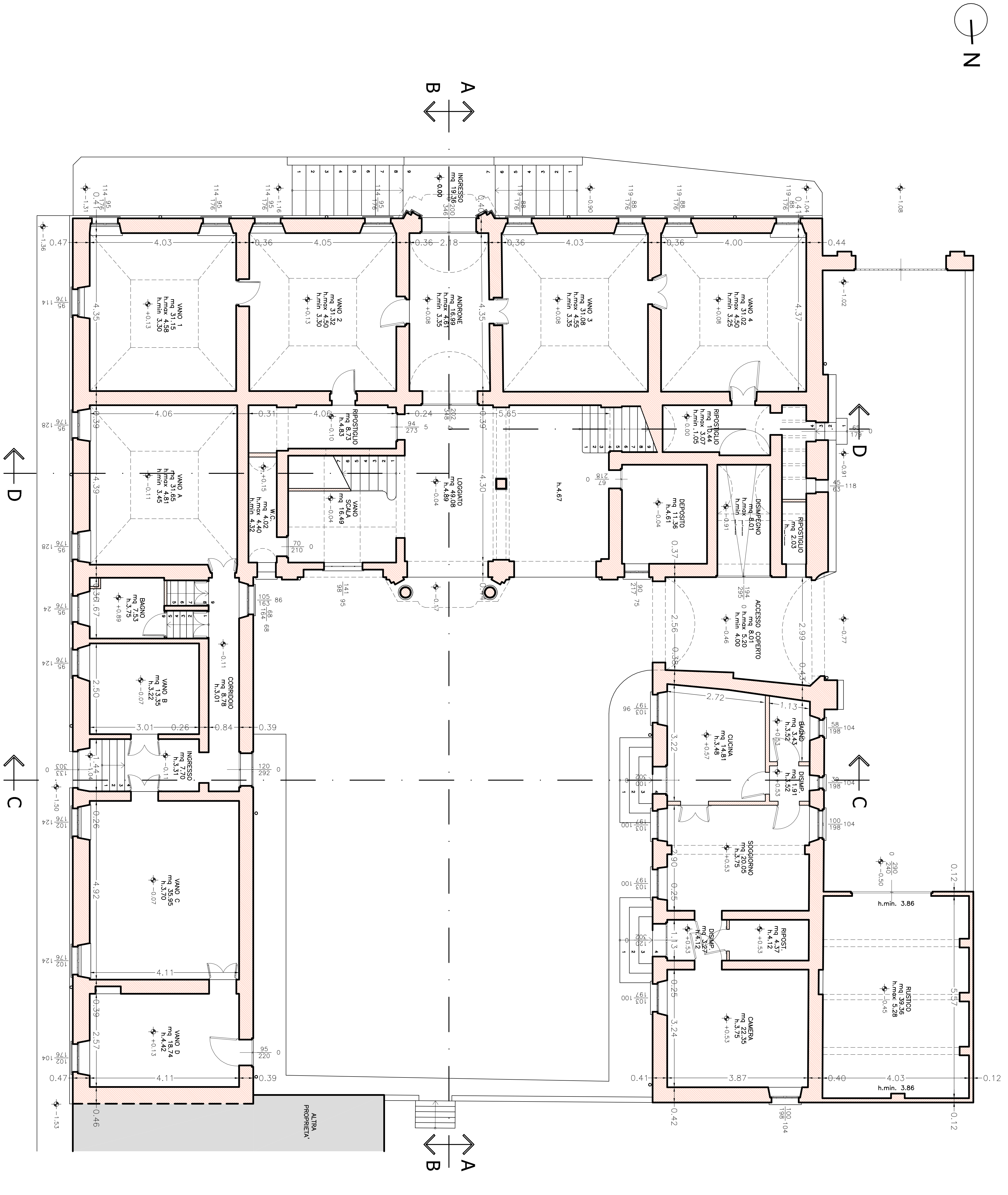
APPENDIX: PLANTS, ELEVATIONS AND SECTIONS

PALAZZO BOLDI
BASEMENT FLOOR PLAN



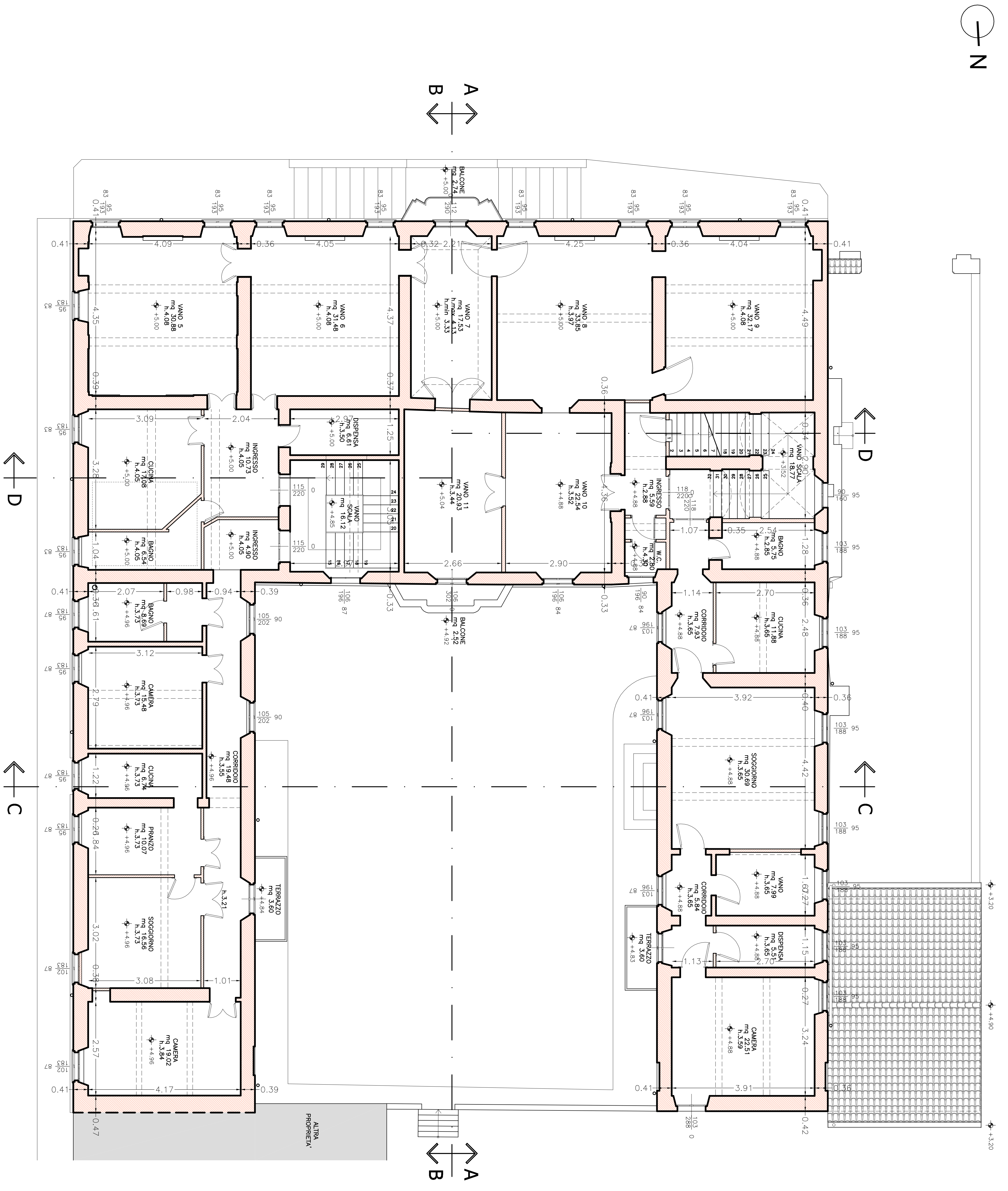
PALAZZO BOLDI

GROUND FLOOR PLAN



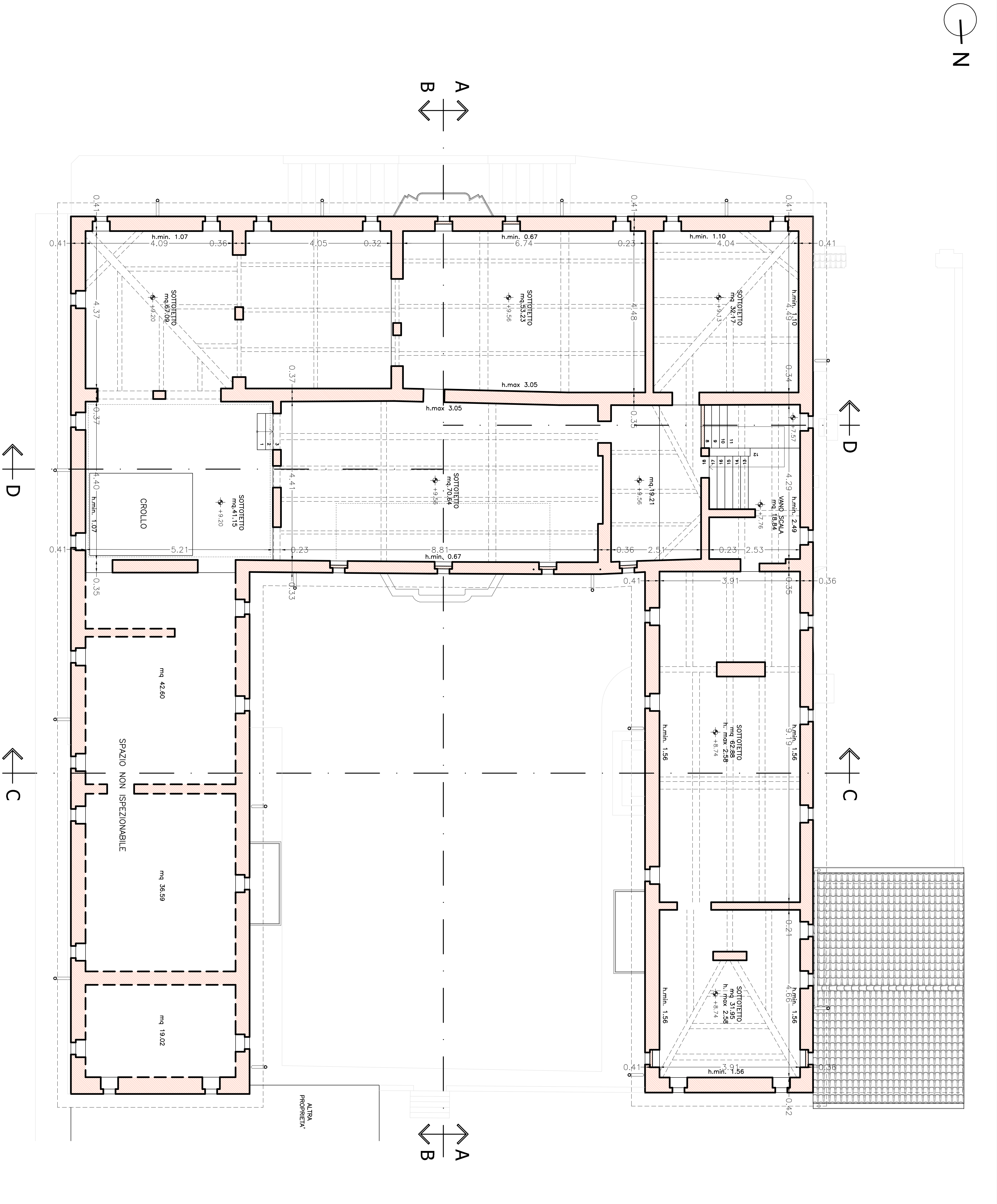
PALAZZO BOLDI

FIRST FLOOR PLAN

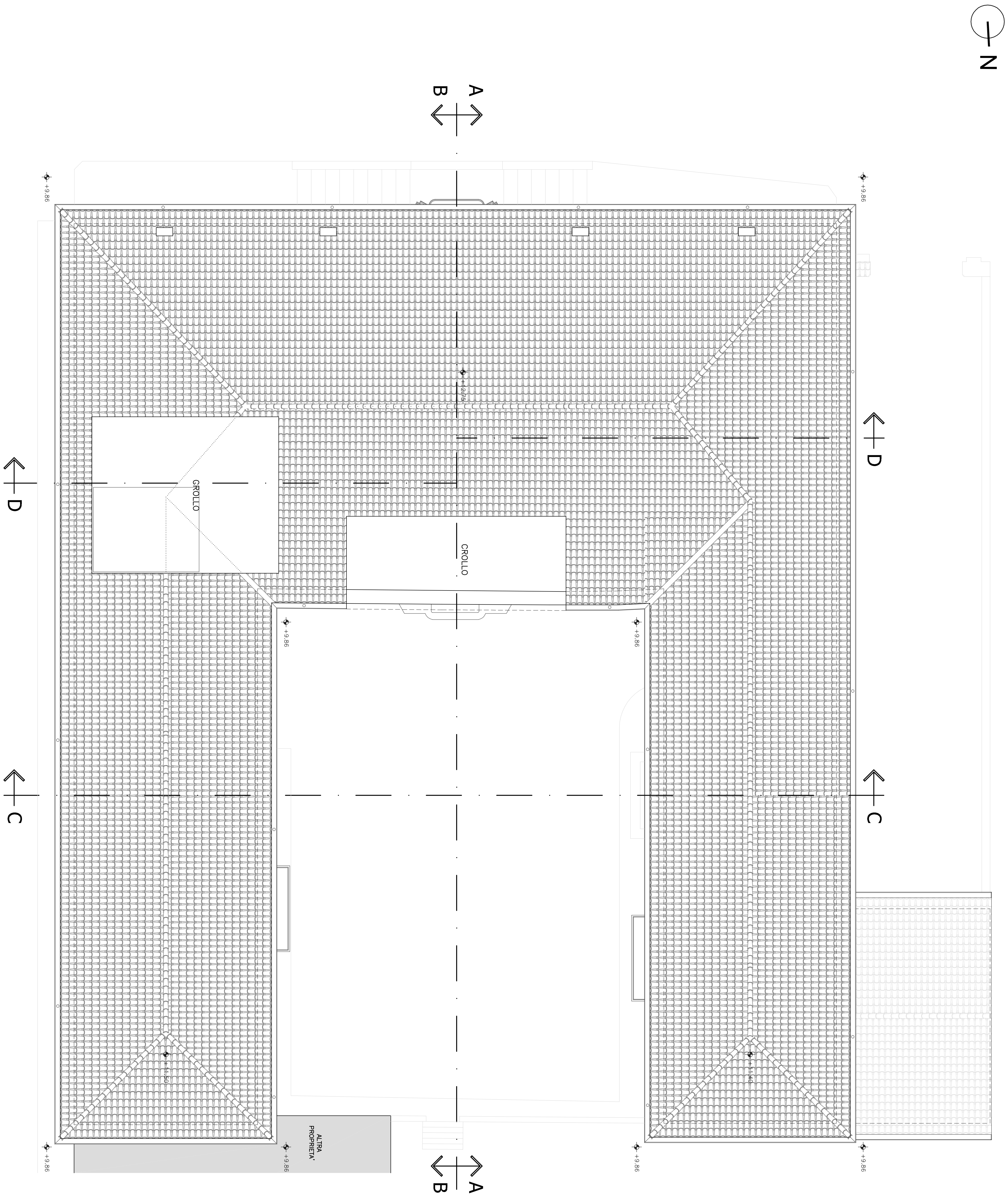


PALAZZO BOLDI

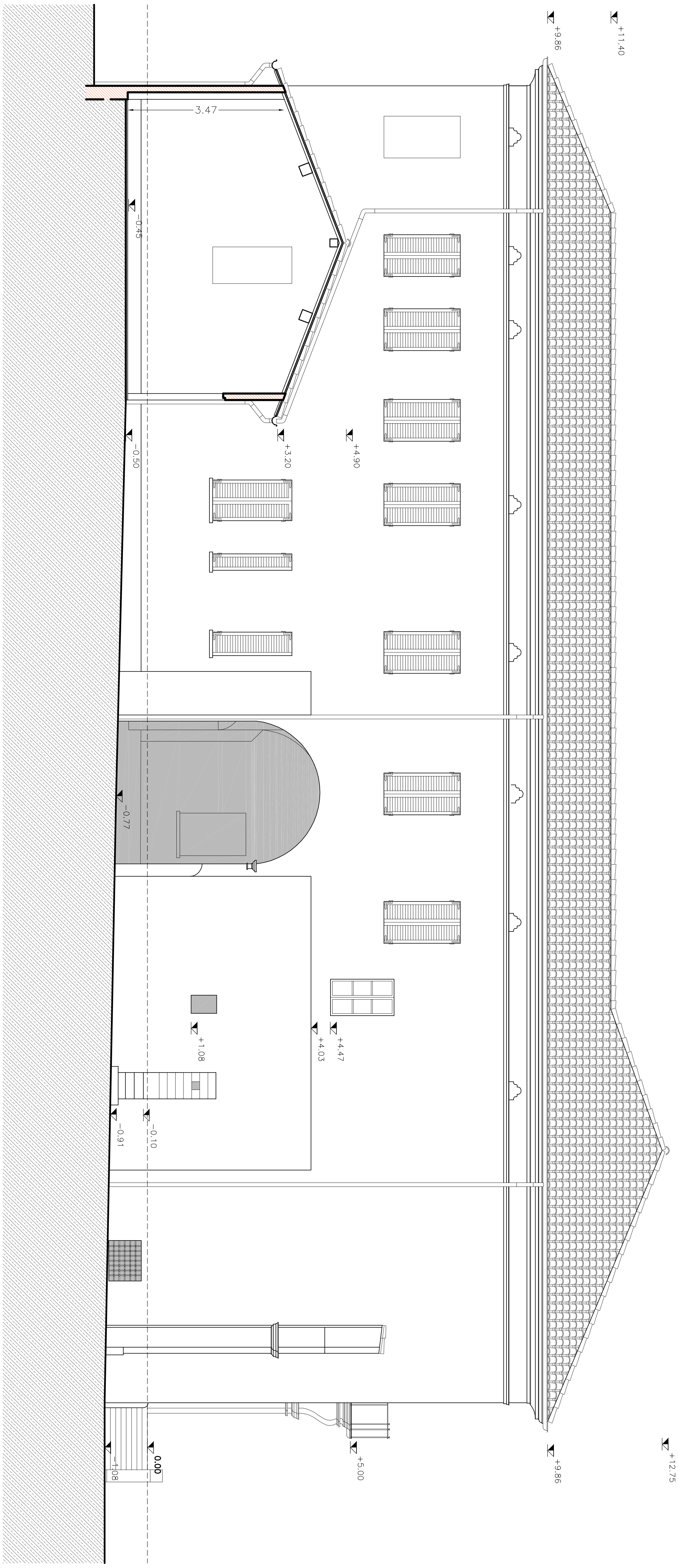
SECOND FLOOR PLAN



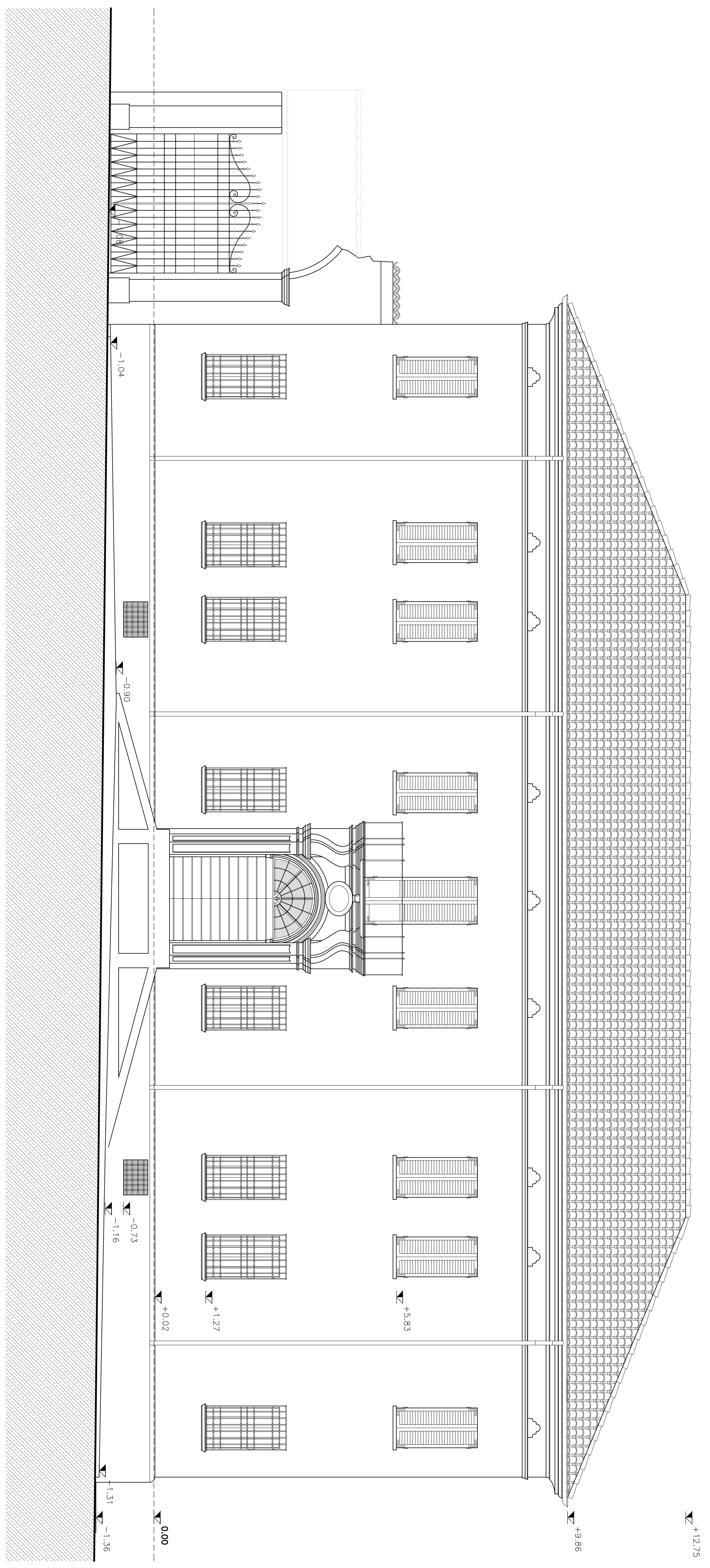
PALAZZO BOLDI
ROOF PLAN



PALAZZO BOLDI
WEST ELEVATION

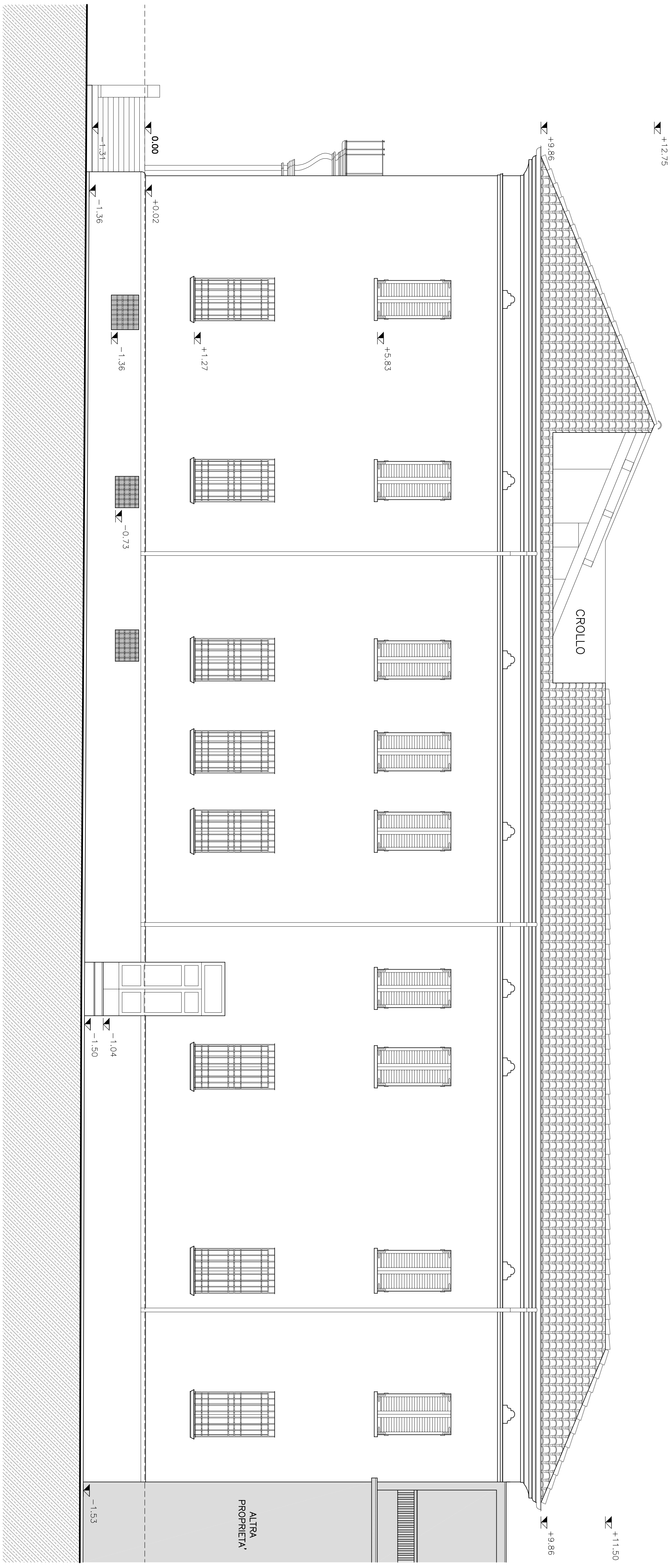


PALAZZO BOLDI
SOUTH ELEVATION



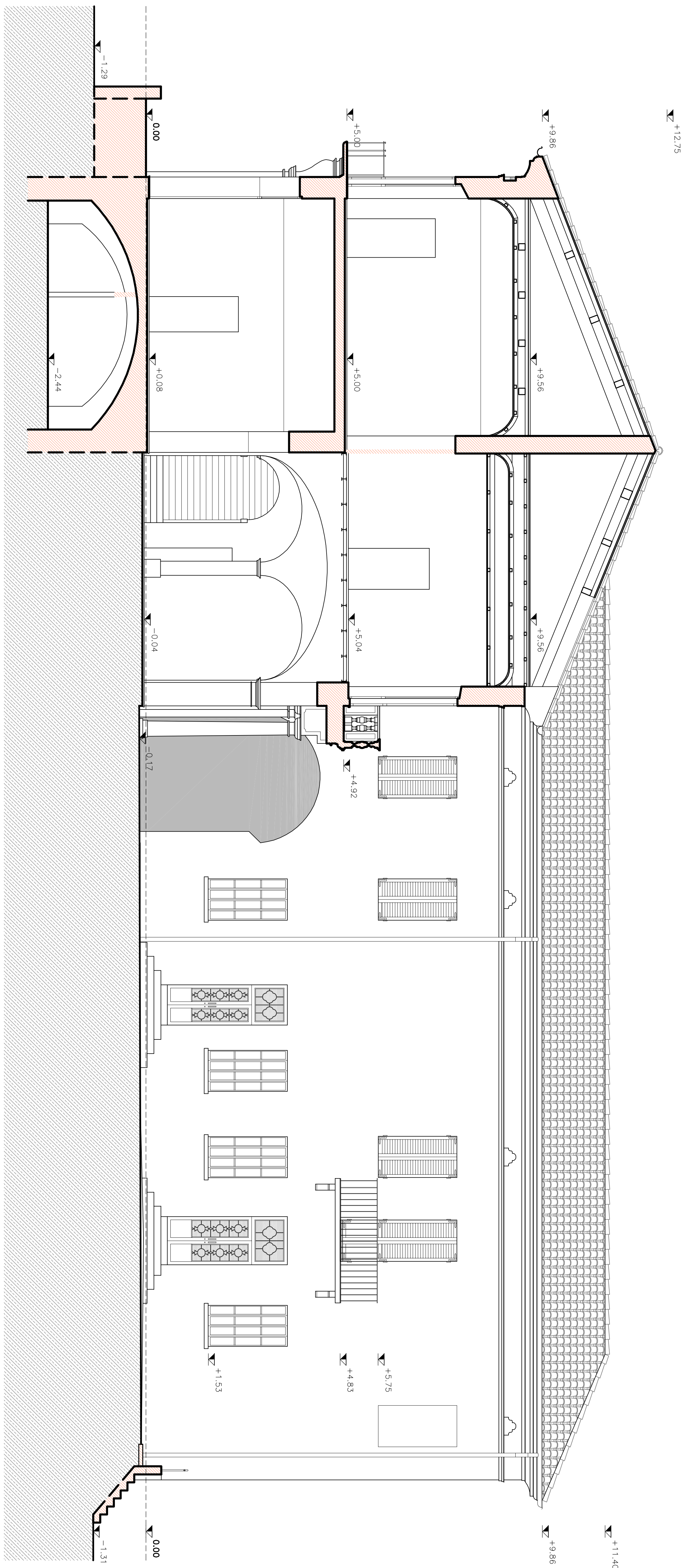
PALAZZO BOLDI

EAST ELEVATION



PALAZZO BOLDI

SECTION A-A



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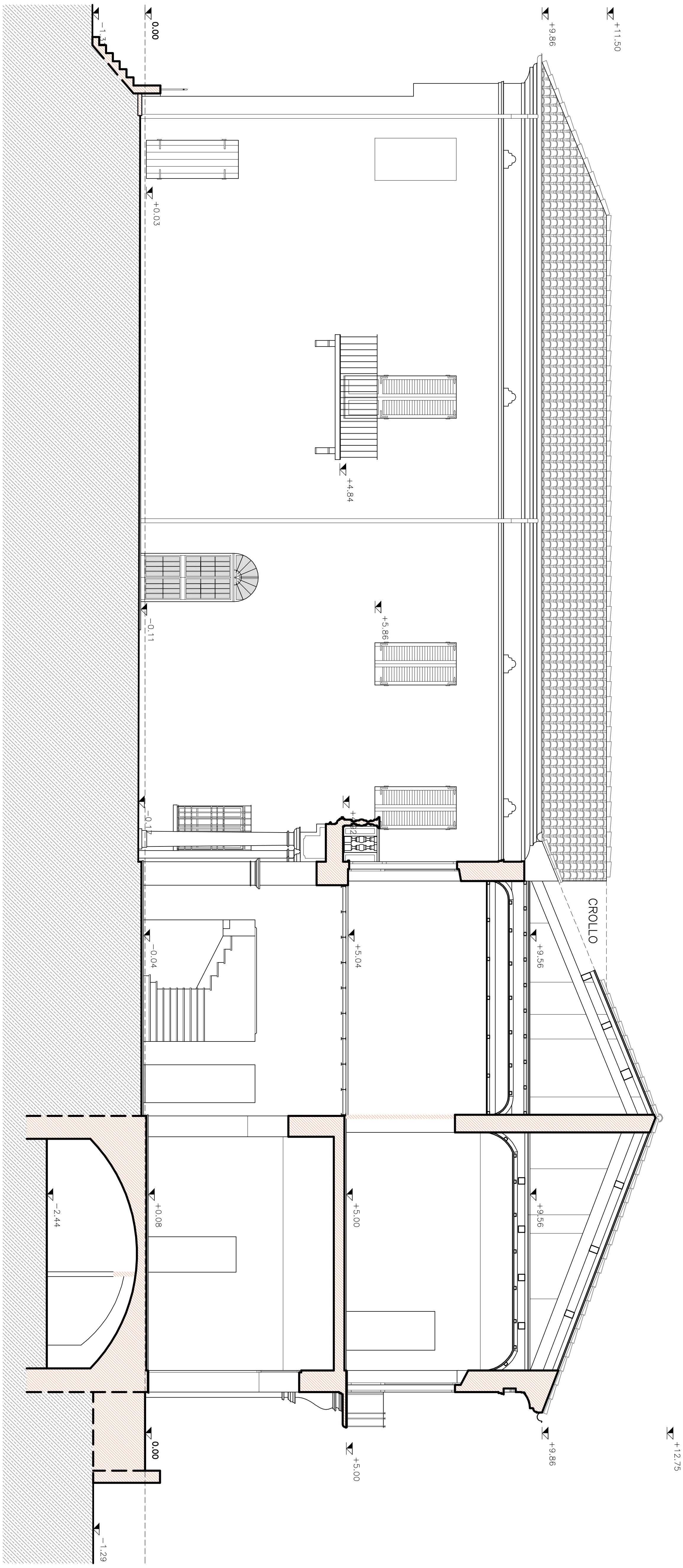
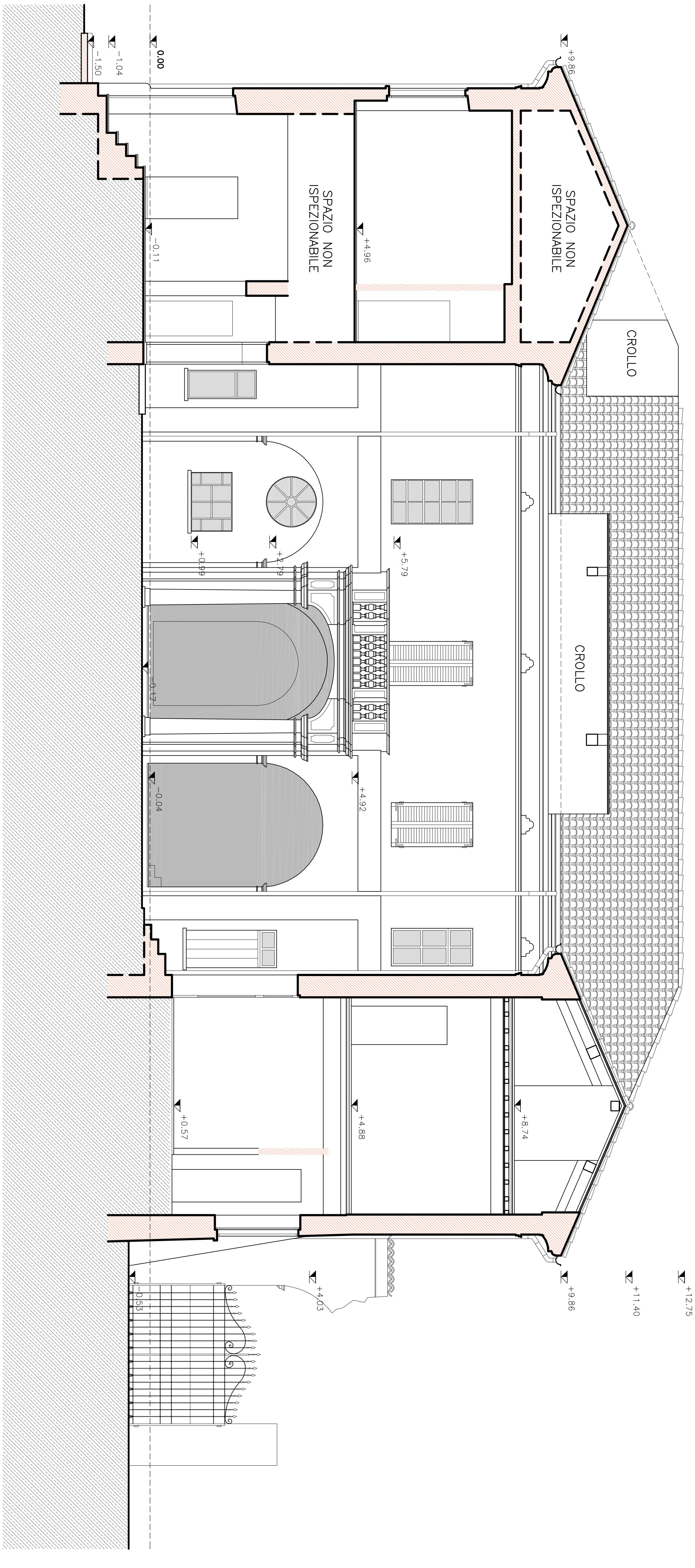
SECTION B-B



PALAZZO BOLDI

SECTION C-C



PALAZZO BOLDI

SECTION D-D