National Technical University of Athens
School of Civil Engineering
Laboratory for Earthquake Engineering

Postgraduate Thesis

VALIDATED LINEAR AND NON-LINEAR ANALYSIS OF MASONRY BUILDINGS THROUGH SHAKING TABLE AND IN-SITU TESTING

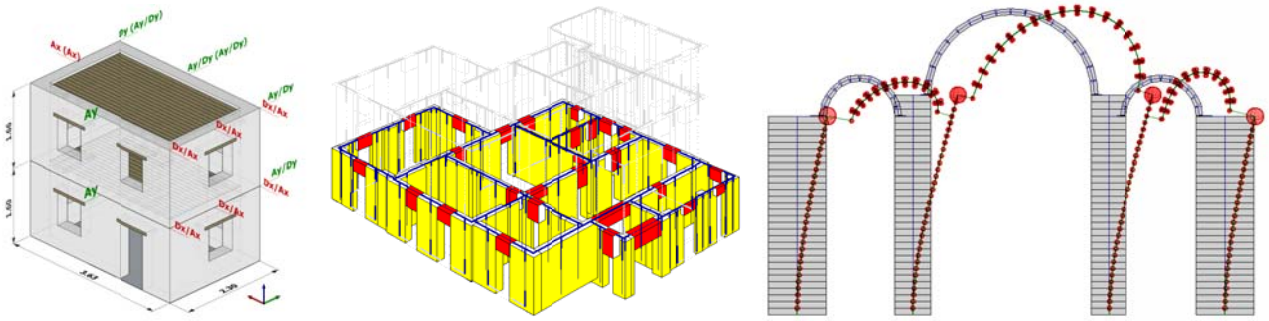

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Table of contents

Introduction

The seismic assessment and retrofit of existing buildings is one of the main topics in the world of constructions due to progressive relative reduction of new construction activity with respect to interventions on existing structures, but the complexity of the topic constitutes a great obstacle to a strict codification of methodologies and approaches.

The first chapter deals with general consideration on the current norms for interventions on masonry building. More specifically a comparison between the Eurocode and the Italian Standards is proposed, highlighting several issues in which the Italian norms felt the necessity to introduce more articulation with respect to the relevant European norm.

Chapter 2 presents linear and non-linear analysis performed on a half scaled building specimen tested at the shaking table of the Laboratory of Earthquake Engineering of NTUA.

The analyses of the building were carried out using three different models developed with well known FEM programs. Through comparison between the experimentally extracted and the calculated modal properties, the accuracy of the FEM models was assessed and non-linear pushover analysis was performed.

Chapter 3 deals with the assessment and retrofit of a three story residential masonry building. Extended in-situ testing and investigations were carried out in order to evaluate the mechanical properties of the materials and the quality of the construction details. Linear and non-linear analyses were performed both at the initial state of the building and after the strengthening interventions. A comparison between the two methods of analyses is proposed and discussed.

Chapter 4 offers a detailed report of the flat jack tests performed in the building analyzed in chapter 3. The procedure of execution and the results of single flat jack test and double flat jack test are presented.

The structural evaluation of a historic orthodox church is the topic of chapter 5. Based on the results of in-situ testing and modal response spectrum analysis previously carried out on the church, the chapter deals with a local non-linear analysis in order to evaluate the transversal seismic response of the church at the initial state and after the proposed strengthening interventions. The methods used for the structural modeling and the results of the analyses will be presented.

2

General consideration on current norms for interventions on masonry buildings

The seismic assessment and retrofit of existing buildings, among which a great number are unreinforced masonry, has become one of the main topics in the world of constructions, also due to progressive relative reduction of new construction activity with respect to interventions on existing structures. The topic itself is extremely complex, due to the enormous variability of structural forms and materials that can be found in countries with a long history of civilization such as in Europe, ranging from simple houses to spectacular monumental structures. Such variety constitutes a great obstacle to a strict codification of methodologies and approaches, as it is possible with new designs. Dealing with masonry buildings, not only the structural forms and materials differ from country to country, but such structural forms very often cannot be approached with the same engineering criteria used for reinforced concrete or steel construction, even when the simpler and regular residential buildings are considered.

Eurocode 8 part 3 (EN 1998-3) deals with the assessment and retrofitting of buildings. The attempt of implementing EN 1998-3 in the new Italian structural standards (NTC2008 and the relevant guidelines Circ.NTC08) revealed, from the Italian point of view, a series of novelties that in part were a serious progress towards a safe and rational approach to the assessment, in part were not compatible with the reality of the problem due to the impossibility to extend to masonry buildings concepts and procedures which would be appropriate for other types of structures such as reinforced concrete or steel framed buildings.

An important step forward coming from EN 1998-3 was the introduction of the fundamental problem of the knowledge of the structure, and the conceptual definition of different knowledge levels and consequent confidence factors for assessment. However, in the rational definition of knowledge levels of masonry buildings it should be noted that in most real cases:

- no construction drawings neither structural design are available, neither test reports;

- the building was built in absence of design regulations, and in the best case conforming to a “rule of art”, so no “simulation of design” is thinkable;
- often the direct experimental measurement of material parameters is not feasible or, if in principle feasible, completely unreliable.

In the Italian Standards NTC2008 it was therefore felt essential to define specific criteria for masonry regarding the different knowledge levels.

Another important issue, in which the Italian norms felt the necessity to introduce more articulation with respect to EN 1998-3, was the definition of the type of intervention in relationship with the increase in safety/performance level that is being pursued (local intervention, improvement/strengthening intervention, retrofit), taking into consideration the situations in which the complete retrofit of the building is not possible.

A considerable expansion and partial correction of the criteria for safety/performance assessment presented in chapter 4 of EN 1998-3 was considered necessary for masonry buildings regarding two main aspects:

- the need to address local mechanisms, often related to out-of-plane response of walls of or
- parts of the structure, which could be approached by ad-hoc methods, and which constitute an essential guidance in the design of the strengthening/retrofit intervention; this issue is not addressed in EN 1998-3;
- the impossibility to apply to masonry buildings the schematic “ductile mechanism/brittle mechanism” conceptual framework which is applied to r.c. or steel structures and the consequent difficulty to apply the methodologies of analysis proposed in EN 1998-3.

At the same time, it was felt that an informative and updated annex on strengthening techniques and strategies would have been an important complement to the norms, in the light of recent post-earthquake experiences in Italy.

The following paragraphs introduce the relevant part of the new Italian structural standards (NTC2008 and relevant guidelines), specifically chapter 8 that deals with existing buildings, highlighting the main novelties with respect to EN1998-3.

1.1 SCOPE

The safety of the existing buildings is of high importance in Italy, so for the high vulnerability, especially with respect to seismic actions, as for the historical and

architectural value of the buildings. At the same time, the large variety of structural typologies arising from the different characteristics of walls and horizontal elements, the presence of ties, anchors and other types of connections in the case of masonry structures, adds further complexity.

Therefore, the assessment methods and the intervention techniques, both traditional and modern, are difficult to be standardized. For this reason, a performance approach has been followed, with the adoption of few general rules and some important hints for the accuracy of analysis, design and execution of the interventions.

These standards apply to existing buildings whose structure is completed at the time of the assessment and the interventions design.

Among others the concepts of knowledge level (related to geometry, construction details and materials) and consequent confidence factor for the assessment are introduced.

The cases for which an assessment is required are defined, for existing buildings it can be carried out checking only the Ultimate Limit States. Specifically, an assessment must be carried out any time a structural intervention is performed and it should determine the safety level of the building before and after the intervention. The designer must report in a specific document the current safety levels and those achieved with the intervention, as well as any consequent limitations to be imposed in the use of the building.

Three categories of intervention are identified: seismic upgrading, seismic improvement and local interventions. The conditions for which the intervention of seismic upgrading is necessary are established, together with the necessity of a static test, for both the interventions of seismic upgrading and improvement.

Some basic steps for safety evaluation and design are defined: historical analysis, geometrical and structural survey, mechanical characterization of materials, definition of levels of knowledge and resulting confidence factors, definition of actions and structural analysis.

Furthermore, the criteria for the use of traditional and modern materials, to repair and strengthen the structures are set. Special attention is given to the assessment and the design in presence of seismic actions, highlighting the peculiarities of masonry structures compared to those of reinforced concrete and steel building.

As regards the existing masonry buildings, a distinction is made between local and global collapse mechanisms, establishing that safety of construction must be assessed for both of them. For buildings in aggregate, especially frequent in historic centers, the criteria for the identification of structural units and for their structural analysis are defined, taking into

account the complexity of the behavior, the inevitable interactions with adjacent structural units and the possible simplifications to the calculation.

In addition, some basic criteria for intervention common to all the typologies are defined, such as the regularity of the interventions, the delicacy and importance of the execution phase and the priority given to the interventions in accordance with the results of the evaluation, primarily to counter the development of local and brittle mechanisms. Furthermore, specific interventions for each structural typology are identified.

Finally, the main steps of a project of seismic upgrading or improvement are defined, starting from the verification of the structure before the intervention, with identification of the structural weaknesses and the level of seismic action for which the ultimate limit state is reached (and serviceability limit state, if required), proceeding with the choice of intervention and techniques to be adopted, the preliminary dimensioning, structural analysis and final verification with determination of the new level of seismic action for which the ultimate limit state is reached (and serviceability limit state, if required).

1.2 GENERAL CRITERIA

Wherever not otherwise specified in this chapter, the general criteria provided in the other chapters of this standard apply also to existing buildings.

In case of non-structural interventions, their effects on the ULS and SLS of the structure or parts of it should be assessed. A typical example of non-structural intervention is the creation or modification of building systems in masonry structures, due to the insertion of pipes in load-bearing walls or the creation of niches which significantly weaken the structural elements and the connection between them. Another example is the demolition of partition or infill walls whose stiffness and strength are not negligible, especially in frame structures. Whenever the structural behavior under vertical and seismic loads is in danger, a structural evaluation is required.

Since existing structures:

- reflect the state of knowledge at the time of their construction;
- possibly contain hidden gross errors;
- may have suffered previous earthquakes or other accidental actions with unknown effects;
- may exhibit degradation and significant changes with respect to the original state;

the structural evaluation and the structural intervention are typically subjected to a different degree of uncertainty (level of knowledge) than the design of new structures. The existence of the structure allows to determine the actual mechanical properties of the materials which can vary considerably within the same structure and cannot be imposed as design data to be achieved during execution, as in case of a new building. On the other hand, a correct and accurate evaluation reduces the uncertainties that in a new building are inherent in the transition from design to construction.

Therefore, in the definition of the structural models of existing buildings, it should be considered that:

- the geometry and the construction details are defined and their knowledge depends on the available documentation and on how detailed are the surveys;
- the knowledge of the mechanical properties of the materials is not affected by uncertainties related to production and installation, but depends on the homogeneity of the materials within the building, the level of detail and the reliability of the surveys;
- permanent loads are defined and their knowledge depends on the level of detail of the surveys.

The verification methods of new buildings are based on partial safety factors which apply to the actions and the mechanical properties of the materials and account for the whole process from design to execution. In existing buildings the knowledge of structure (geometry, construction details and materials) is crucial. That is why another category of factors is introduced: the *confidence factors,* which reduce the average strength of the materials according to the relevant knowledge level.

1.3 STRUCTURAL EVALUATION

The structural evaluation is a quantitative procedure aimed to:

- determine whether an existing undamaged or damaged structure will withstand the design actions;
- determine the maximum value of the actions considered in the design combinations, that the structure is able to withstand with the required safety margin defined by the partial safety factors on actions and materials.

Whenever possible, the method used should incorporate information of the observed behavior of the same type of building under similar actions. This applies especially when the evaluation is performed with respect to seismic actions.

The structural evaluation and the design of the interventions on existing buildings may be carried out in terms of Ultimate Limit State. If an evaluation in terms of Serviceability Limit State is also carried out, the performance requirements can be set by the designer together with the client. The evaluation with respect to the ULS can be carried out checking the Limit State of Life Safeguard (Significant Damage for Eurocode). The Limit State of Collapse (Near Collapse for Eurocode) is considered only for concrete and steel buildings and it can be checked as an alternative to the Limit State of Life Safeguard.

A structural evaluation of the existing building must be performed in case of:

- considerable reduction of strength or deformation capacity of the structure due to environmental actions (earthquake, wind, snow and temperature), significant degradation and decay of the mechanical properties of the materials, exceptional actions (impacts, fires, explosions), anomalous use, significant deformations due to sinking foundations;
- serious errors in design or construction;
- change of the intended use of the building, with significant change of the variable loads;
- non-structural interventions that reduce the capacity or alter the stiffness of the structural elements.

If the cases referred to in the previous list involve limited portions of the building, the structural evaluation can be limited to the elements involved and those interacting with them taking into account their role in the structure as a whole.

The structural evaluation must allow to determine whether:

- the use of the building can go on without interventions;
- the use should be changed (change of the intended use or imposition of use restrictions);
- the load-bearing capacity should be enhanced or restored.

The structural evaluation shall determine the safety level before and after the interventions. The designer must report in a specific document the current safety levels and those achieved with the intervention.

In case of non-compliance of the verification with respect to permanent loads and serviceability actions, the necessary measures must be applied. More complex is the situation in case of non-compliance with respect to environmental actions subject to big variations and uncertainties in their determination. For obvious reasons, we cannot think of imposing mandatory interventions or change of intended use, as soon as a non-compliance occurs. The decisions must be taken case-by-case according to the gravity of the non-

compliance, the consequences, the economic availability and implications in terms of public safety. The owners or the managers of each construction, public or private, shall define the most appropriate measures in order to increase safety, taking into account the nominal life and the class of use of the structure.

In protected buildings, the improvement interventions are generally able to combine the needs of conservation with those of safety. However, for such buildings, interventions that clearly alter the structural configuration or require the execution of invasive works, as in case of enlargements or superelevation, should be avoided.

1.4 CLASSIFICATIONS OF THE INTERVENTIONS

The following categories of intervention are identified:

- *Seismic upgrading*, aimed to pursue a determined safety level according to the standard requirements,
- *Seismic improvement*, aimed to increase the existing safety level even without achieving the standard requirements,
- *Local interventions*, which involve single structural elements and improve the existing safety levels.

The interventions of seismic upgrading and seismic improvement require static testing. Regardless of the three categories identified, the interventions should primarily aim at the elimination or reduction of serious deficiencies related to errors in design and execution, degradation, damages and transformations.

1.4.1 Seismic upgrading

Among the seismic upgrading interventions appear the following interventions:

- addition of stories over an existing building;
- enlargements of the building;
- change of intended use of the building that leads to an increase in terms of global load;
- introduction of independent and important structures into existing constructions.

In any case, the design should refer to the building as a whole and should include the evaluation of the entire structure before and after the interventions.

The structural evaluation must state whether the structure, after the intervention, can withstand the design actions defined in the Standard with the required safety level. In general, it is not necessary to meet the requirements on construction details (minimum

reinforcement, stirrup spacing, minimum size of beams and pillars, etc.) which apply for new constructions, provided that the Designer proves that performances in terms of resistance, ductility and deformability are guaranteed in any case.

1.4.2 Seismic improvement

Among the seismic improvement appear all the interventions aimed to increase the resistance of the existing structures under the design actions that are not considered seismic upgrading: interventions that vary significantly stiffness, strength and ductility of single elements or introduce new structural elements, so to modify the local or global behavior of the structure particularly with respect to seismic actions. Obviously, the changes must improve the structural behavior, for instance better exploiting the most resistant elements, reducing irregularities in plan and elevation, switching the brittle collapse mechanisms to ductile.

The structural evaluation is mandatory and shall determine the highest value of the design actions that the structure can withstand with the required safety level. The design and structural evaluation should involve all the parts of the structure whose structural behavior is likely to be affected, as well the structure as a whole.

1.4.3 Local interventions

Among the local interventions appear:

- repair, strengthening and replacement of single structural elements (beams, lintels, slab portions, columns, walls) provided that the intervention do not change significantly the overall behavior of the structure through a significant change in stiffness or weight, especially with respect to seismic resistance;
- replacement of roofs and slabs, provided that the intervention does not lead to a significant change of in-plane stiffness, important for the distribution of the horizontal forces, or to an increase of the vertical loads;
- restoration or strengthening of the connections between structural elements such as wall-to-wall or wall-to-floor connections and the introduction of ties.

The design and the structural evaluation shall refer just to the elements involved and shall state that, compared to the previous configuration, the behavior of the structure as a whole did not change and that the interventions led to an improvement of the existing safety level.

The structural report shall highlight the weaknesses identified, both solved and persistent, and shall indicate any consequent restriction of use of the building.

1.5 INFORMATION FOR STRUCTURAL ASSESSMENT

Given the large variety of typologies detectable among the existing structures, it is impossible to provide specific rule for each case. Therefore, the Designer should chose the method for the structural evaluation, case-by-case in relation to the structural behavior of the building, taking into account the following general information.

1.5.1 Historical analysis

In order to evaluate the existing structural system and the actual state of stress it is important to know the historical evolution of the building and to collect information on possible structural changes since construction.

Generally, dealing with existing structures, it may be difficult to find the original design drawings and reconstruct the historical evolution. Sometimes, especially for cultural heritage structures, it is possible to collect information complete enough on their structural evolution through researches in the archives. In any case, especially for masonry structures, it is advisable to collect information on the historical evolution of the area where the structure is located, referring to specific documents.

The historical analysis shall allow to determine how many earthquakes the structure has suffered in the past. This is very important because allows to evaluate the seismic behavior of the building, provided that in the meanwhile the structural configuration and the mechanical characteristics of the materials did not change significantly.

On the base of the information collected in the historical analysis, useful hints can be drawn for the structural modeling of the building.

1.5.2 Geometrical survey

An important step towards the acquisition of the data for an accurate modeling is the geometrical survey of the structure. The survey shall be performed on the structure as a whole as well as on the single structural elements. It shall individuate the load-bearing system, taking into account the quality and the condition of the structural elements and the constituent materials. Information about the type and extent of previous and present structural damages, including earlier repair measures shall be also collected.

The geometrical survey shall be supported by drawings such as plans, elevations and sections of the building as well as construction details.

1.5.3 Mechanical characterization of the materials

An adequate knowledge of the mechanical characteristics of the materials and their decay can be achieved through the available documentation, on-site investigations and laboratory tests. The survey shall be planned showing the reasons and the goals of each investigation. In case of cultural heritage structures special attention must be paid to the impact of the investigation in terms of preservation of the structure.

If several reliable investigations were performed on the structure, the mechanical characteristics of the materials can be drawn from them regardless of the categories provided by the Standard.

A non-exhaustive hint for the definition of the mechanical characteristics of the materials can be found in the Standards in force at the time of construction.

1.6 KNOWLEDGE LEVELS

Depending on the amount and quality of the information collected on the points above, the knowledge levels of the different factors involved in the model (geometry, details, materials) shall be defined as well as the relevant confidence factors to be adopted as partial safety factor accounting for the lack of knowledge of the building.

1.6.1 Geometry

The knowledge of the structural geometry of existing masonry buildings comes from the survey of all the masonry elements including niches, cavities, chimneys, the survey of the vaults, the slabs , the roof, the stairs, the foundations and the definition of the loads acting on each element.

Especially important are the survey and the drawings of the crack patterns, each crack shall be classified according with the related mechanism (detachment, rotation, sliding, etc.). This is of great help in understanding the state of damage of the structure, its possible causes and the type of survey to be performed.

1.6.2 Details

The construction detail to be examined are the following:

a) quality of the connection between walls;
b) quality of the connection between floors and walls;
c) presence of lintels above the openings;
d) presence of structural elements aimed at counteract possible thrusts;
e) presence of vulnerable elements;
f) masonry typology(single-leaf, three-leaf, with or without filling, with or without diatones).

The following methods may be adopted in the inspections:

- *limited in-situ inspections*: are based on the visual surveys of the walls after having removed the plaster. They allow to examine the characteristics of the wall on the surface and to assess the connection between orthogonal walls and between floors and walls. The construction details of points a) and b) may be evaluated even on the base of the masonry typology.
- *extended and comprehensive in-situ inspection*: are based on the visual surveys of the walls after having removed the plaster and took masonry samples. They allow to examine the characteristics of the wall both on the surface and along its thickness and to assess the connection between orthogonal walls and between floors and walls. The inspection are carried out in a comprehensive way throughout the building.

1.6.3 Materials

In the evaluation of the quality and the mechanical properties of masonry particularly important are the shape, the typology and the dimension of the masonry units, the pattern in which the units are assembled, the quality of the mortar and the workmanship. Significant is also the characterization of the mortar (type of binder, type of aggregate, binder/aggregate ratio, carbonation level) and the bricks or stones through laboratory tests. Mortar and units are sampled in-situ. The following tests can be carried out:

- *Limited in-situ testing*: is a procedure for complementing the information on material properties derived either from standards at the time of construction, or from original design specifications, or from original test reports. It is based on the visual survey of the masonry surface after having removed a rectangular area of plaster of about 1m x 1m, in order to determine shape and dimension of the units. Such survey shall preferably be performed at the corners in order to evaluate the quality of the connection between the walls. The consistency of the mortar shall be even approximately evaluated.

- *Extended in-situ testing*: the investigations referred to above shall be carried out extensively taking samples for each typology of masonry. Double flat jack tests and laboratory test on mortars and masonry units allow to determine the masonry typology. At least one test shall be performed for each masonry type. Several non-destructive tests (sonic test, ponder drilling test, penetration test for the mortars, etc.) may be used in addition to the test required. In case of a clear typological analogy for materials, pattern and details, the tests performed on nearby buildings may be used for the building under analysis.
- *Comprehensive in-situ testing*: is a procedure for obtaining information on the resistance of the materials. In addition to visual surveys, samples, and the investigation referred to above, further tests are performed in order to evaluate the mechanical characteristics of masonry. The measurement of the mechanical characteristics of masonry is obtained through in-situ tests or laboratory tests on samples taken from the structures. In general the tests may include diagonal compression tests or combined tests of vertical compression and shear force. Non-destructive methods may be applied in combination but not as an alternative to those referred to above. The results of the tests shall be considered within a general typological framework that takes into account the test reports available in the literature and allows to assess the actual validity of the results. Therefore, the test results shall be used in combination with Table 1 as described in the following paragraph.

1.6.4 Knowledge Levels

The following three knowledge levels are defined:

- *KL1 Limited knowledge:* is achieved if the geometrical survey, limited in-situ inspections on the construction details and limited in-situ testing on the materials have been carried out. The corresponding confidence factor is CF = 1.35;
- *KL2 Normal knowledge:* is achieved if the geometrical survey, extended and comprehensive in-situ inspections on the construction details and extended in-situ testing on the materials have been carried out. The corresponding confidence factor is CF = 1.2;
- *KL3 Full knowledge:* is achieved if the geometrical survey, extended and comprehensive in-situ inspections on the construction details and comprehensive in-situ

testing on the materials have been carried out. The corresponding confidence factor is CF = 1

Based on the knowledge level achieved, for each masonry typology the mean value of the mechanical properties shall be defined as follows:

- **KL1**

  Strength: minimum value of the range given in Table 1.

  Modulus of elasticity: mean value of the range given in Table 1.

- **KL2**

  Strength: mean value of the range given in Table1.

  Modulus of elasticity: mean value of the range given in Table1.

- **KL3**

  Strength:

  - three or more test results are available: the average of the test results shall be used;
  - two test results are available: if the average of the test results falls into the range given in Table 1, the mean value of the range shall be used; if the average falls outside the range, the minimum or the maximum value of the range shall be used;
  - one test result is available: if the test result falls into the range given in Table 1 or it is higher, the mean value of the range shall be used; if the test result is lower than the range, the test results itself shall be used.

  Modulus of elasticity: average of the test results or mean value of the range given in Table1

1.7 MASONRY TYPOLOGIES AND RELEVANT MECHANICAL PROPERTIES

Table 1 shows the reference values for the different masonry typologies, that can be used in the analysis according to the knowledge level as described above.

Table 1. Reference values of the mechanical characteristics of different masonry typologies. f_m is the mean compression strength; $\tau 0$ is the mean shear strength; E is the mean value of the elastic modulus; G is the mean value of the shear modulus; w is the mean specific weight.

Masonry typology	**f_m (N/cm^2)**	**τ_0 (N/cm^2)**	**E (N/mm^2)**	**G (N/mm^2)**	**W (kN/m^3)**
	min - max	**min - max**	**min - max**	**min - max**	
Rubble masonry	100 180	2.0 3.2	690 1050	230 350	19
Rough-hewn stone masonry	200 300	3.5 5.1	1020 1440	340 480	20

Dressed stone masonry	260 380	5.6 7.4	1500 1980	500 660	21
Soft stone masonry	140 240	2.8 4.2	900 1260	300 420	16
Ashlar masonry	600 800	9.0 12.0	2400 3200	780 940	22
Solid brick masonry	240 400	6.0 9.2	1200 1800	400 600	18
Cored bricks masonry (void area ≤ 40%)	500 800	24.0 32.0	3500 5600	875 1400	15
Cored bricks masonry (void area < 45%)	400 600	30.0 40.0	3600 5400	1080 1620	12
Cored bricks masonry with dry vertical joints (void area ≤ 45%)	300 400	10.0 13.0	2700 3600	810 1080	11
Concrete block masonry (45% ≤ void area ≤ 65%)	150 200	9.5 12.5	1200 1600	300 400	12
Concrete block masonry (void area < 45%)	300 440	18.0 24.0	2400 3520	600 880	14

In the case of historic masonry, the values in Table 1 (first six typologies) refer to masonry in bad condition, bad quality mortar, mortar joints not particularly thin, absence of elements aimed to regularize the pattern. Moreover the values refer to masonry without artificial diatones.

The values provided for regular masonry refer to cases in which the pattern is workmanlike. In case of incorrect patterns (mortar joints not well staggered) the values must be adequately reduced.

If the masonry has better characteristics than the one described above, the coefficients given in Table 2 shall be applied to the mechanical values given in Table 1:

- good quality mortar: the coefficient given in Table 2 for the different typologies shall be applied both to strength (f_m and τ_0) and modulus of elasticity (E and G);
- thin mortar joints (<10 mm): the coefficient given in Table 2 for the different typologies shall be applied both to strength (f_m and τ_0) and modulus of elasticity (E and G); in the case of the shear strength the increase in percent shall be the half of the one considered for the compressive strength;
- presence of elements aimed to regularize the pattern: the coefficient given in Table 2 shall be applied just to the strength (f_m and τ_0);

- presence of artificial diatones: the coefficient shall be applied just to the strength (f_m and τ_0); such coefficient is applicable just in case of historic masonry;

The different typologies listed in Table 1 refers to three-leaf masonry with relatively thin internal leaf. If the internal leaf is relatively big with respect to the external leaf, the mechanical properties of masonry shall be adequately reduced by means of the coefficient given in Table 2.

In case of consolidated masonry the mechanical characteristics for some intervention techniques can be evaluated through the coefficients given in Table 2. The coefficients can be applied both to strength (f_m and τ_0) and modulus of elasticity (E and G).

Table 2. Corrective coefficients of the mechanical properties given in Table 1.

Masonry typology	Good quality mortar	Thin joints	Regular pattern	Artificial diatones	Wide internal leaf	Grout injection	Reinforced concrete jacket
Rubble masonry	1.5	-	1.3	1.5	0.9	2	2.5
Rough-hewn stone masonry	1.4	1.2	1.2	1.5	0.8	1.7	2
Dressed stone masonry	1.3	-	1.1	1.3	0.8	1.5	1.5
Soft stone masonry	1.5	1.5	-	1.5	0.9	1.7	2
Ashlar masonry	1.2	1.2	-	1.2	0.7	1.2	1.2
Solid brick masonry	1.5	1.5	-	1.3	0.7	1.5	1.5

1.8 SEISMIC ASSESSMENT

In existing masonry structures subject to seismic actions, local and global failure modes may occur. The local mechanisms affect the out-of-plane response of single wall panels or wider portions of the structure and occur when the connections between orthogonal walls and between walls and floors are particularly poor. The global mechanisms affect the structure as a whole and are governed by the in-plane response of the walls. The safety of the structure must be evaluated with respect to both failure modes.

The seismic analysis in terms of local mechanisms can be performed using limit analysis taking into account, even approximately, the compression strength of masonry, the quality

of the connections and the presence of ties. With such method it is possible to evaluate the seismic capacity in terms of resistance (applying an appropriate behavior factor) or displacement.

The global seismic analysis shall consider the real structural system of the building paying particular attention to the stiffness of the slabs and the effectiveness of the connection between structural elements. In case of irregular masonry, the in-plane shear resistance of the walls can be calculated by mean of alternative formulations with respect to the ones used for new structures.

In case of aggregate buildings, adjacent or interconnected to other buildings, the verification methods used for new buildings may not be adequate. In the analysis of such buildings a structural unit shall be identified taking into account the interactions arising from the continuity with the adjacent buildings.

The structural unit shall have vertical continuity from the roof to the foundations and, normally, shall be delimited by open spaces, structural joints or contiguous building of different typologies. In addition to the provisions for non-aggregate buildings, the following aspects shall be evaluated: thrusts of adjacent slabs against common walls, local mechanisms of the façade due to adjacent structural units of different heights.

In the global analysis of a single structural unit simplified methodologies may be used. The assessment of a structural unit with rigid slabs may be performed by mean of non-linear static analysis, analyzing and checking each story of the building separately and neglecting the variation of axial force in the walls due to the seismic action. If the slabs of the structural unit can move only in one direction, the analysis can be performed neglecting the torsional effects. Instead, in case of structural units located on the corner, simplified methodologies can still be used provided that the torsional effects are accounted for by mean of relevant coefficients that increase the horizontal actions.

If the slabs of the building are flexible the single walls or the systems of coplanar walls may be analyzed separately, provided that for each wall the relevant vertical load and in-plane seismic action are defined.

1.8.1 Performance requirements

The structural evaluation of existing masonry structures requires the check of the following limit state:

Limit State of Life Safeguard (Significant Damage). The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of

sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.

In addition to the global analysis of the building, an analysis of the local mechanisms must be performed. The building seismic response can be governed by such mechanisms when connections between orthogonal walls and between walls and floors are particularly poor. This is often the case in existing stone masonry buildings without tie rods and ring beams, with lack of interlocking at the connection of intersecting walls, presence of simply supported wooden floors and thrusting roofs. In this cases a global analysis can be performed as an exhaustive set of analyses of the local mechanisms, provided that the seismic action is distributed among them taking into account their interactions.

1.8.2 Seismic action and seismic load combination

The seismic action for the Limit State of Life Safeguard(Significant Damage) is defined in the relevant part of this Standard. For the assessment of buildings with linear analysis and q-factor approach, the following values must be adopted:

- $q = 2.0\ \alpha_u/\alpha_1$ for buildings regular in elevation
- $q = 1.5\ \alpha_u/\alpha_1$ in the other cases

where: α_1 is the multiplier of the horizontal design seismic action at the formation of the first plastic hinge in the system; α_u is the multiplier of the horizontal seismic design action at the formation of a global plastic mechanism.

The value of α_u/α_1 can be calculated by mean of non-linear static analysis and in any case not exceeding 2.5. In the absence of more accurate evaluations, the value of α_u/α_1 can be assumed equal to 1.5.

The design seismic action shall be combined with the other appropriate permanent and variable actions in accordance with the relevant part of this Standard.

1.8.3 Structural modeling

The stiffness of the walls shall be calculated considering both the flexural and shear stiffness. Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-half of the corresponding stiffness of the uncracked elements.

In general the structure may be considered to consist of a number of vertical and lateral load resisting systems, connected by horizontal diaphragms. When the floor diaphragms of the building may be taken as being rigid in their planes, the structural model may include just the vertical element continuous for all the height of the structure and connected at the floor levels.

Otherwise, the elements connecting different walls, such as concrete or masonry beam (if they are adequately connected to the walls), may be included in the structural model provided that the safety verifications are carried out even for these elements.

Masonry coupling beams may be considered in the model only if they feature a bending resistant lintel adequately anchored to the adjacent walls. Concrete elements may be considered as coupling elements if their height is at least equal to the thickness of the slab.

In the presence of coupling beams the analysis may be performed using an equivalent frame model where the intersection between vertical and horizontal elements may be considered infinitely rigid.

In case of rigid floors the analysis-determined shear distribution among the wall panels of the same story may be modified provided that the global equilibrium is preserved and that the absolute value of the shear variation in each wall panel is

$$\Delta V \leq \max\{0.25|V|, 0.1|V_{storey}|\}$$

where: V is the shear in the wall panel and V_{storey} is the total shear in the storey and the direction of the wall panel.

In case of flexible floors the shear can be redistributed within panels of the same wall and V_{storey} refers to the sum of the shear in each panel of the wall.

The mean values of material properties shall be used in the structural model. In case of linear analysis the strength of the materials is calculated dividing the mean values by the relevant confidence factor and partial safety factor. In case of non-linear analysis the strength is calculated dividing the mean values by the confidence factor.

1.8.4 Methods of global analysis

The seismic action effects, combined with the effects of the other permanent and variable loads in accordance with the seismic load combination, may be evaluated using one of the following methods:

- static linear analysis (lateral force)

- modal response spectrum analysis
- non-linear static analysis (pushover)
- non-linear dynamic analysis

1.8.4.1 Static linear analysis

The static linear analysis is based on the application of static forces equivalent to the inertial forces caused by the seismic action. This type of analysis may be applied to buildings whose fundamental period (T_1) of vibration in a given direction does not exceed 2.5 T_C o T_D.

For buildings with heights of up to 40 m the value of T_1 (in s) may be approximated by the following expression:

$$T_1 = C_1 \cdot H^{3/4}$$

where:

H is the height of the building in m, from the foundation or from the top of a rigid basement;

C_1 is 0.085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures.

The seismic base shear force Fb, for each horizontal direction in which the building is analyzed, shall be determined using the following expression:

$$F_b = S_d(T_1) \cdot m \cdot \lambda$$

where:

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

$S_d(T_1)$ is the ordinate of the design spectrum at period T_1;

m is the total mass of the building, above the foundation or above the top of a rigid basement;

λ is the correction factor, the value of which is 0,85 if $T_1 < 2T_C$ and the building is regular in elevation and has more than two storeys, or 1.0 otherwise.

The distribution of the forces follows the fundamental mode shape of the structure approximated by horizontal displacement increasing linearly along the height of the building:

$$F_i = F_b \cdot \frac{z_i \cdot W_i}{\sum_j z_j W_j}$$

where:

F_i is the horizontal force acting on mass mi;

F_b is the seismic base shear;

z_i, z_j are the heights of the masses m_i and m_j above the level of application of the seismic action;

W_i, W_j are the weights of the masses m_i and m_j.

If the lateral stiffness and mass are symmetrically distributed in plan, the accidental torsional effects may be accounted for by multiplying the action effects in the individual load resisting elements by a factor δ given by

$$\delta = 1 + 0.6 \cdot \frac{x}{L_e}$$

where:

x is the distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered;

L_e is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

1.8.4.2 Modal response spectrum analysis

The analysis is based on:

- the definition of the vibration modes of the structure (modal analysis);
- the calculation of the seismic action effects for each mode of the structure;
- the combination of the modal responses.

The response of all modes of vibration contributing significantly to the global response shall be taken into account. All modes with effective modal masses greater than 5% of the total mass should be taken into account and the sum of the effective modal masses for the modes taken into account should amount to at least 85% of the total mass of the structure.

For the combination of the modal responses a Complete Quadratic Combination shall be adopted as in the following formula:

$$E = \left(\sum_j \sum_i \rho_{ij} \cdot E_i \cdot E_j \right)^{1/2}$$

where:

E is the seismic action effect under consideration;

E_j is the value of this seismic action effect due to the vibration mode j;

ρ_{ij} is the correlation coefficient between mode i and mode j given by

$$\rho_{ij} = \frac{8\xi^2 \beta_{ij}^{3/2}}{(1+\beta_{ij})\left[(1-\beta_{ij})^2 + 4\xi^2\beta_{ij}\right]}$$

ξ is the viscous damping;

β_{ij} is the inverse of the ratio of the periods $(\beta_{ij} = T_j/T_i)$

The accidental torsional effects may be determined from the application of static loadings, consisting of sets of torsional moments about the vertical axis of each storey:

$$M_{ai} = e_{ai} \cdot F_i$$

where:

M_{ai} is the torsional moment applied at storey i about its vertical axis;

e_{ai} is the accidental eccentricity of storey mass i;

F_i is the horizontal force acting on storey i.

1.8.4.3 Non-linear static (pushover) analysis

Pushover analysis is a non-linear static analysis carried out under constant gravity loads and a system of horizontal loads whose sum is represented by the base shear F_b. The horizontal loads, applied at each floor of the structure along the direction considered, shall monotonically increase in both positive and negative directions until a global or local collapse occurs. The relation between base shear force and the control displacement, which may be taken at the centre of mass of the last floor of the structure, represents the capacity of the structure.

Masonry and concrete elements may feature an elastic - fully plastic behavior, with ultimate resistance equal to the elastic limit and ultimate displacement defined by the flexural and shear capacity.

The analysis may be applied for the following purposes:

- to evaluate the overstrength ratio values α_u/α_1;
- to assess the effective plastic mechanisms and the distribution of damage in building designed with behavior factor q;
- as an alternative to the design based on linear-elastic analysis;
- to assess the structural performance of existing buildings.

At least two vertical distributions of the lateral loads should be applied:

- a "modal" pattern, based on lateral forces that are proportional to mass and elevation;
- a "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation.

The target displacement shall be defined as the seismic demand derived from the elastic response spectrum in terms of the displacement of an equivalent single-degree-of-freedom system.

1.8.4.4 Non-linear dynamic analysis

The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using a accelerograms to represent the ground motions.

The analysis aims to evaluate the non-linear dynamic behavior of the structure, allowing for a comparison between the ductility demand and available ductility.

The non-linear dynamic analysis shall be compared with a modal response spectrum analysis in order to check the differences in terms of global base reactions.

In case of structure with seismic base isolation the non-linear dynamic analysis is mandatory when the isolation system cannot be represented by an equivalent linear model.

1.8.5 Safety verifications

In case of linear analysis, the safety verification in terms of ultimate limit state is satisfied if for all structural elements the resistance in terms of in-plane bending, in-plane shear and out-of-plane bending is greater than the corresponding design value of the action effect, due to the seismic design situation. The out-of-plane bending resistance has to be checked for all the load-bearing walls even if they are not considered seismic resistant.

In case of non-linear static analysis, the safety verifications is based on the comparison between the ultimate displacement of the structure and the displacement demand.

1.9 STRUCTURAL INTERVENTIONS

The strengthening interventions should be applied as much as possible regularly and uniformly on the building, so to avoid uneven distributions of strength and stiffness. Eventual increase of these factors on limited portions of the building must be carefully evaluated. In addition particular care must be paid to the execution phase, since a bad execution may worse the global behavior of the structure.

The choice of the most appropriate approach depends on the results of the previous evaluation phase, with the target to counteract the local mechanism and improve the global behavior of the structure.

In general, the following aspects should be taken into account:

- possible existing damages should be repaired;
- all identified local gross errors should be appropriately remedied;
- in case of highly irregular buildings (both in terms of stiffness and overstrength distributions), structural regularity should be improved as much as possible, both in elevation and in plan;
- reduction of the masses through partial demolitions or change of use of the building;
- reduce horizontal diaphragm deformability;
- improve the connections with non-structural elements;
- increase the resistance of the vertical structural elements, taking into account that the increase in strength should not reduce the available global ductility;
- construction, improvement or demolition of seismic joint;
- improvement of the foundations where required;
- improvement of the connections between orthogonal walls and between walls and slabs;
- reduction of the thrusts of roofs, arches and vaults;
- strengthening of the walls next to openings.

2

Shaking table test and pushover analysis on a scaled masonry building

During an earthquake both out-of-plane and in-plane response are simultaneously mobilized, but it is generally recognized that a satisfactory seismic behavior is attained only if out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully exploited. A global model of the structure is usually needed when the resistance of the building to horizontal actions is provided by the combined effect of floor diaphragms and in-plane response of structural walls [6].

The building specimen studied in this chapter was tested at the shaking table for two different configurations. Specifically, at its initial state, the diaphragm constructed of wooden planks and beams was rather flexible while, after testing that caused damages to the structure, it was stiffened to achieve "rigid type" behavior. A comparison of the experimental results for both states of the building is presented.

Modal and non-linear analysis of the building have been carried out using three different models developed with well known FEM programs. Since the role of non-linear static analysis is progressively recognized as a practical tool to evaluate the seismic response of structures, such analysis was carried out and two different approaches have been pursued [3,4,5]. In the first approach thin shell finite elements were employed to model the masonry walls utilizing non-linear constitutive law. In the second approach an "equivalent frame" model was used, also utilizing the same mechanical characteristics of the materials [7,10].

Through comparison between the experimentally extracted and the calculated modal properties, the accuracy of the FEM models was assessed. Once the accurate FEM simulation was validated, non-linear pushover analysis was performed. A comparison between the results obtained with the shell element modelling approach and those obtained with the equivalent frame approach is presented [9].

2.1 DESCRIPTION OF THE SPECIMEN

The specimen analyzed is a two-storey half-scaled masonry building constructed at the Laboratory of Earthquake Engineering (LEE) of the National Technical University of Athens (NTUA). It was tested at the shaking table of the laboratory as a part of the European research project NIKER [14].

Figure1 shows the floor plan and a photo of the specimen prior to testing on the shaking table. One door and five windows were arranged along the perimeter of the building at the ground floor, whereas six windows were opened at the first floor.

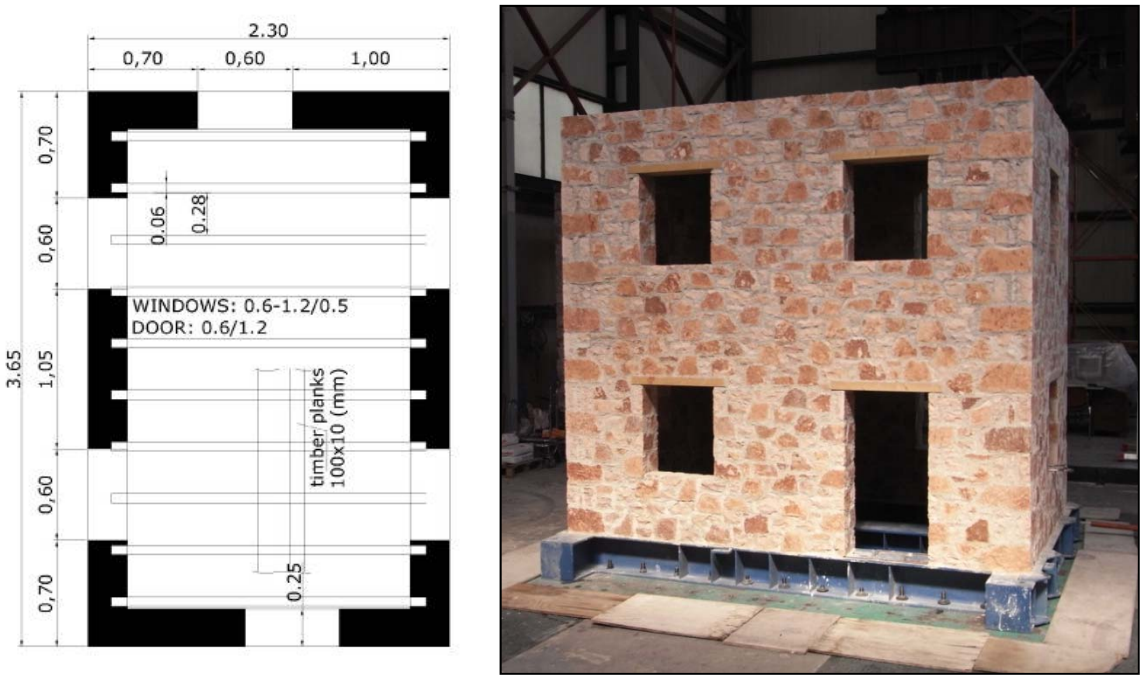


Figure 1. Floor plan and photo of the specimen on shaking table

The specimen was constructed on a steel base to simulate fixed boundary conditions. Also, the base allowed to move the specimen on rollers in order to facilitate the transfer to and from the shaking table.

The walls were made of three-leaf masonry. Limestone with a compressive strength in the order of 100 MPa was used for the construction. The stones thickness did not exceed 10 cm. The building was constructed with a mortar containing a low percentage of cement in order to reduce the curing time needed before testing the specimen.

As shown in Fig.2, the floors consist of timber beams (60x100 mm) placed every 340 mm. Timber beams are placed on collection beams constructed along the walls on the perimeter of the specimen. Also, a timber pavement constructed of timber planks (100x10 mm) nailed to the beams was provided. At the top of all openings timber lintels were placed. For

similitude reasons, additional masses were placed over the floors; in particular, 4.5 Mgr were placed on the first floor and 3 Mgr were placed on the second floor.

The specimen building was subjected to simultaneous seismic motions acting along the two main horizontal axes. As a result the parts of the masonry between openings were subjected to combined in-plane and out-of-plane actions. The geometry of the building allowed formulation of squat and slender piers, a specimen design that allowed to study both flexural as well as shear behavior.

Figure 2. Photos of the wooden slabs and the lintels

Two series of tests were carried out on this specimen:

- *Building at initial state.* A proper protocol for earthquake simulation tests was applied: namely, low acceleration tests to extract the dynamic properties of the model and seismic action tests with increasing acceleration up to considerable damage or failure. For this purpose, an appropriate accelerogram of a real earthquake was selected, scaled and applied in a sequence in order to create damages that could be repairable. After the development of substantial damages, the specimen was strengthened and re-tested.
- *Building with grouted masonry and enhanced diaphragm action of the floors.* After the completion of the test, the model was removed from the shaking table. Grouting was applied to masonry walls using a hydraulic lime based grout. Furthermore, the diaphragm action of the floors was enhanced by placing additional layers of timber planks on the top of the existing pavement. The added timber planks were inclined at 45° from those of the initial pavement and properly fixed with nails. The specimen was subjected to a series of tests similar to the first one, that is: “sweep” in order to extract the modal properties after seismic testing. For comparison, some of the seismic loads of the model at its initial state were repeated. The imposed seismic action was gradually increased, until progressively the specimen developed severe damage.

2.2 INSTRUMENTATION AND EXPERIMENTAL RESULTS

The instruments to measure accelerations (A) and displacements (D) were placed on the specimen following the scheme shown in Fig.3. Eight accelerometers and six displacement transducers were attached to the first floor, whereas seven accelerometers and six displacement transducers were placed at the second floor.

The dynamic properties of the building that is, the natural period of vibration and damping ratio, were extracted through sweep tests applying to the shaking table accelerations of increasing frequency along the longitudinal x-direction and the transverse y-direction. The results are presented in Table 1.

Table 1. Modal properties of the specimen

	Direction	Natural Frequency (Hz)	Natural Period (s)	Damping ratio (%)
At initial state	X	6.0	0.167	5.0
	Y	4.5	0.222	7.0
After interventions	X	10.5	0.095	6.5
	Y	10.5	0.095	5.8

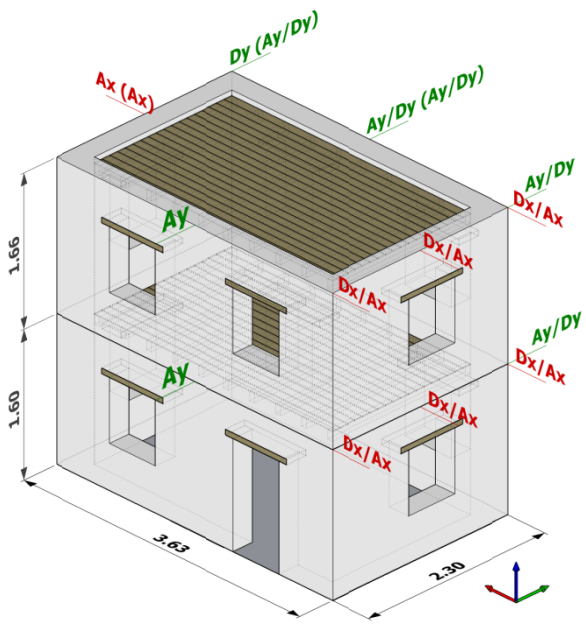


Figure 3. Scheme of the instrumentations

Acceleration time histories were developed based on records from a real earthquake. The spectra of this earthquake, that is, Kalamata (Greece) 1986, along the x and y axis are shown in Fig.4. The acceleration time histories were scaled and applied in a sequence in order to provoke gradual damages to the specimen.

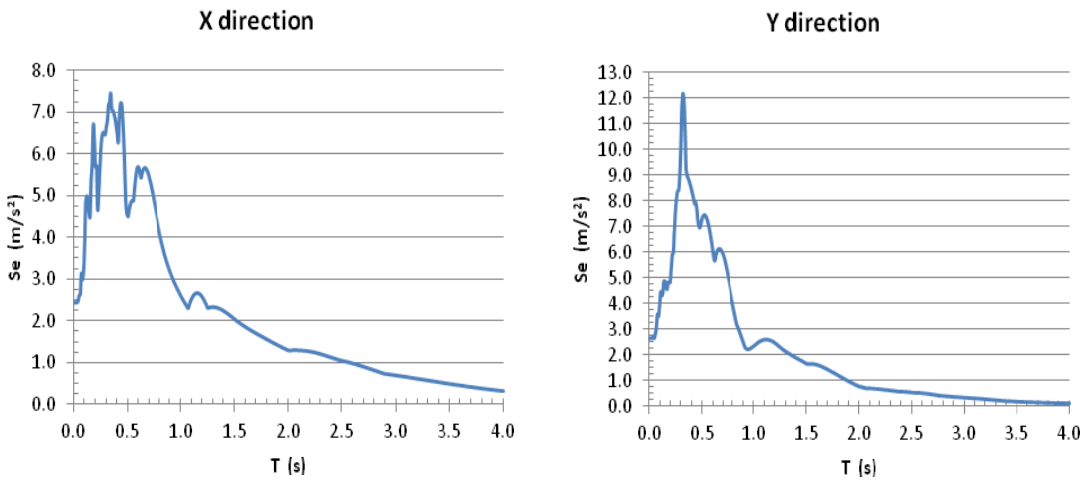


Figure 4. Response spectra of Kalamata earthquake, Greece 1986

2.3 NUMERICAL ANALYSIS

Three different models were used to run modal and static non-linear analysis for the specimen at its initial state as well as after the interventions. The FEM programs used to create the models and perform the analyses were SAP2000 [11], Acca Edilus [12] and Aedes PC.E [13].

2.3.1 Model with SAP2000

The masonry walls were modeled using four-node shell elements. Frame elements were used to model the lintels over the windows as well as the timber beams and the timber planks constituting the floors. The final model shown in Fig.5 consists of 3170 shell elements, 1010 frame elements and 3666 nodes. The timber beams of the floors are pin-connected to the shell elements of the walls and the nodes at the base are fixed. In the model after the interventions a diaphragm action has been imposed to the nodes of the floors in order to account for stiffening in their plane.

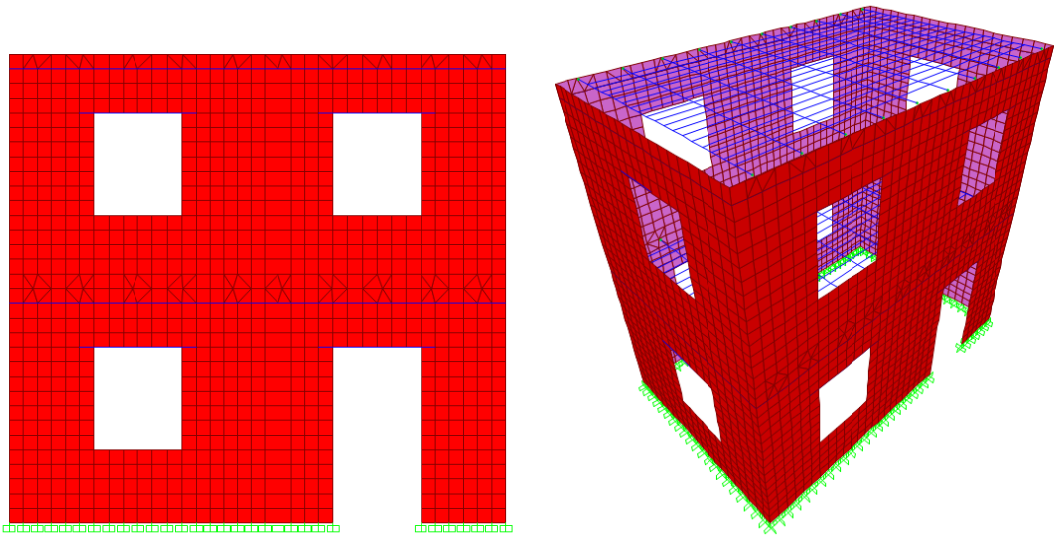


Figure 5. Front view and 3d view of the model with SAP2000

Since the results from compression tests at LEE on half-scaled wallets were not completed, proper mechanical properties of the masonry were selected and shown in Table 2. These properties were used to analyze the specimen at the initial state and after the interventions. In particular, the value of the modulus of elasticity was found by trial-and-error trying to match the natural period of the structure measured with the experiment. The values shown in Table 2 were used in all the other models.

Table 2. Mechanical properties of the masonry

	Modulus of elasticity (MPa)	Shear modulus (MPa)
At initial state	200	80
After interventions	450	180

Table 3 shows the natural mode of vibrations in the horizontal directions of the specimens. For each mode, the period and the participating mass ratio are listed.

Table 3. Modal properties of the model with SAP2000

	Direction	Period (s)	Ux	Uy
At initial state	X	0.166	0.738	0.000
	Y	0.227	0.000	0.752
After interventions	X	0.092	0.867	0.000
	Y	0.111	0.000	0.870

2.3.2 Model with Acca Edilus

The masonry walls were modeled using three-node shell elements, whereas frame elements were used for the lintels and the timber beams. No finite elements were used to model the

timber pavements. Instead a slab object, a special provision of the particular software, spanning from one beam to another was employed in order to collect the load from the floors and distribute it to the adjacent beams. According to program capabilities, such slab object has no stiffness for the building at its initial state, whereas it has infinite stiffness for the case after interventions in order to account for the rapid diaphragm action of the floors. The model shown in Fig.6 consists of 7982 shell elements, 52 frame elements and 4690 nodes. The timber beams of the floors are pin-connected to the shell elements of the walls and the nodes at the base are fixed.

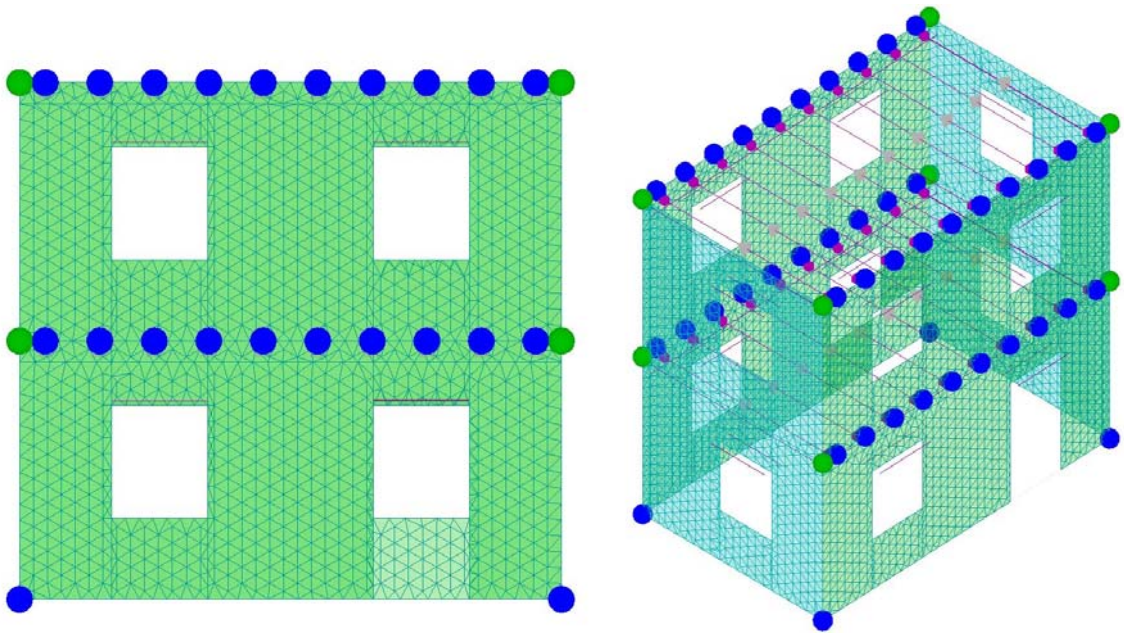


Figure 6. Front view and 3d view of the model with Acca Edilus

First a modal analysis was conducted. Table 4 shows the natural mode of vibrations in the horizontal directions of the building. For each mode the period and the participating mass ratio are presented. The values are quite close to the ones obtained with the experiment. The only exception is the natural period of the building at the initial state along the Y axis (0.310s versus 0.222s). This difference can be attributed to the way the slabs were modeled: the timber pavement was defined only as a load applied to the beams which are the only ones that contribute to the stiffness of the slab, whereas in the model with SAP2000, that better calculate the corresponding period, the slabs were modelled with more detail.

Table 4. Modal properties of the model with Acca Edilus

	Direction	Period (s)	Ux	Uy
At initial state	X	0.174	0.797	0.000

	Y	0.310	0.000	0.617
After interventions	X	0.108	0.829	0.000
	Y	0.127	0.000	0.829

2.3.3 Model with Aedes PC.E

The model is based on an equivalent frame idealization of the structure. Masonry walls are simulated with pier, spandrel and joint elements. The pier and the spandrel elements are frame elements with shear deformation, while the joint elements are infinitely stiff and modeled by means of rigid offsets placed at the ends of the pier and the spandrel elements. No finite elements used to model the floors. Specially, mono-directional slab objects, a special feature of the particular software, spanning from one wall to the other collect the load and distribute it to the walls were employed. For the building at its initial state they have no stiffness, while they are infinitely stiff for the building after the interventions. The model shown in Fig.7 consists of 59 frame elements and 40 nodes. The nodes at the base are fixed.

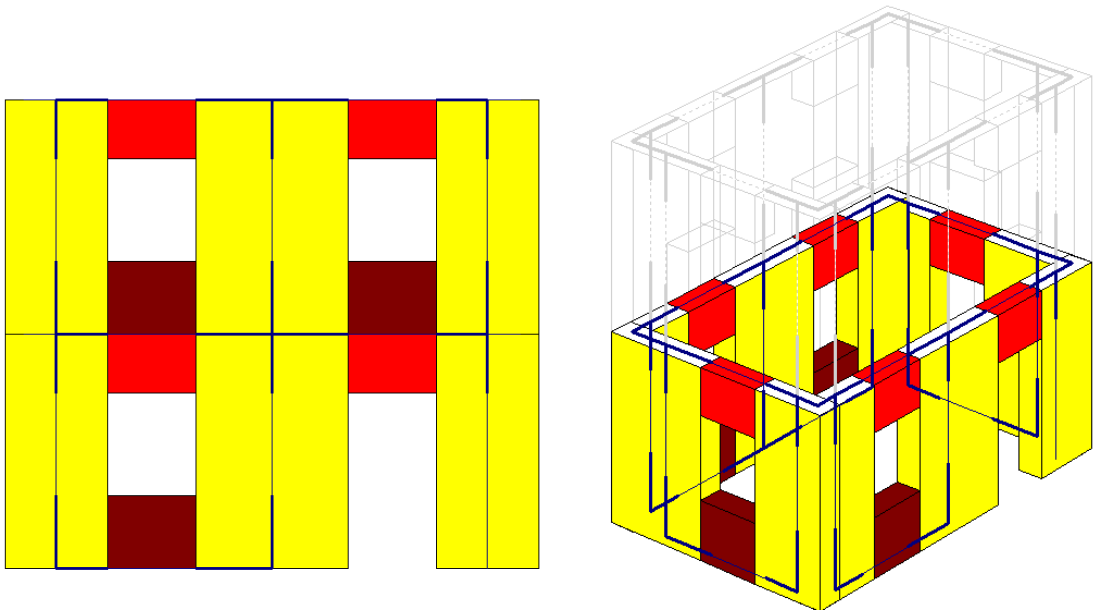


Figure 7. Front view and 3d view of the model with Aedes PC.E

A modal analysis was conducted. Table 5 shows the fundamental mode of vibrations along the horizontal directions of the building. For each mode the period and the participating mass ratio are presented. As for the model with ACCA Edilus, contrary to the other values that are quite close to the experimental results, the natural period of the building along the

y-axis shows a considerable difference. Even in this case the difference from the experimental result (0.222 sec) can be attributed to the way the slabs were modelled.

Table 4. Modal properties of the model with Aedes PC.E

	Direction	Period (s)	Ux	Uy
At initial state	X	0.162	0.806	0.000
	Y	0.308	0.000	0.539
After interventions	X	0.106	0.819	0.000
	Y	0.143	0.000	0.805

2.3.4 Static non-linear analysis

Static non-linear analyses were carried out using the equivalent frame model (PC.E) and the shell elements model (Edilus). A uniform load distribution has been considered along the positive X and Y directions. Figure 8 shows the pushover curves obtained with the equivalent frame model, while the pushover curves obtained with the shell elements model are shown in Fig.9. In each figure the continuous curve refers to the building at the initial state, while the dashed one corresponds to the building after the interventions.

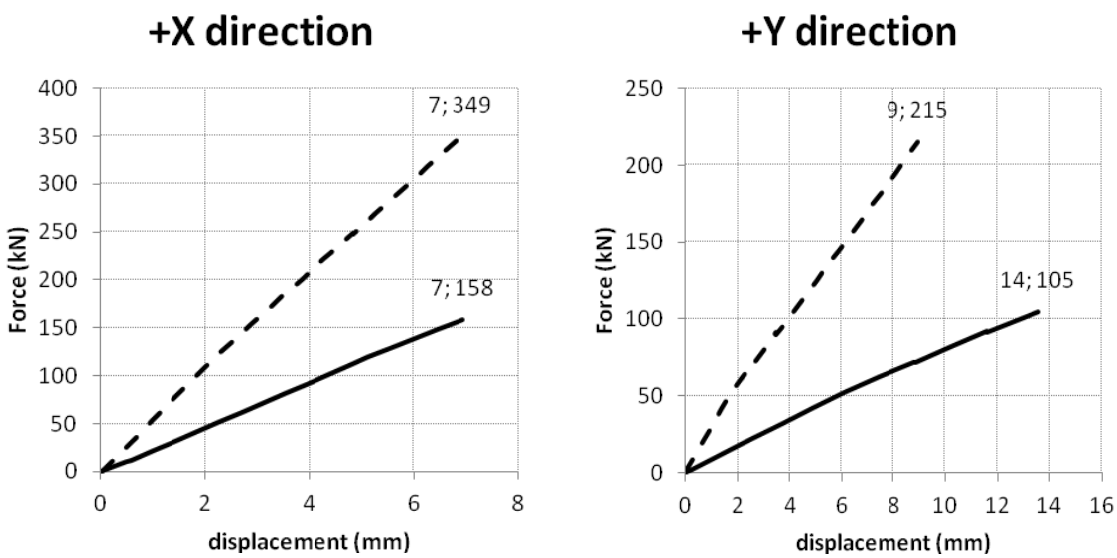


Figure 8. Capacity curves obtained with Aedes PC.E. a) x-direction. b) y-direction

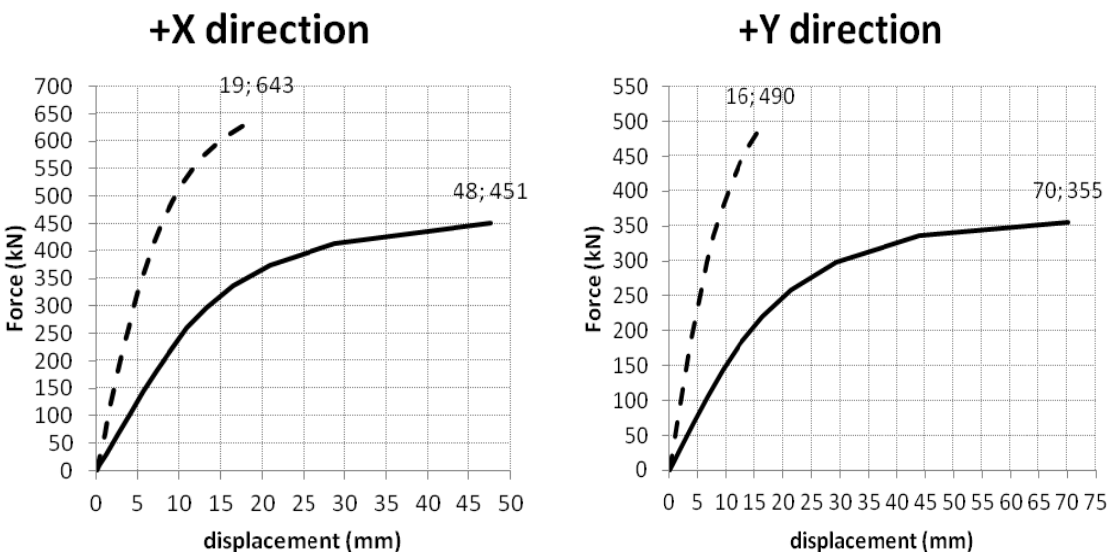


Figure 9. Capacity curves obtained with Acca Edilus. a) x-direction. b) y-direction

2.4 CONCLUSIONS

Based on the comparison between the experimentally extracted and the analytically determined modal properties, it can be stated that all models provided similar results and in close agreement.

Regarding static non-linear analysis, by observing Fig.8a and Fig.9a it is clear that in the PC.E model a "failure" of the specimen occurs at a smaller displacement than the one predicted by the Edilus model. However, both software provide practically the same base shear at the ultimate displacements of the PC.E capacity curves.

A similar observation for the maximum displacements along the Y-axis can be made regarding their values. However, the corresponding base shear differ by almost 100%. At this stage of the research project we were not able to explain this difference. Hopefully, this issue will be clarified when all experiments and associated analyses are completed.

4

Linear and non-linear analysis of an existing residential masonry building

The structure analyzed is a three-story residential masonry building located in Kifisia (Athens). Figure 1 shows the South and East elevations of the building, while Figure 2 shows the ground floor plan.

Figure 1. South elevation and East elevation of the building

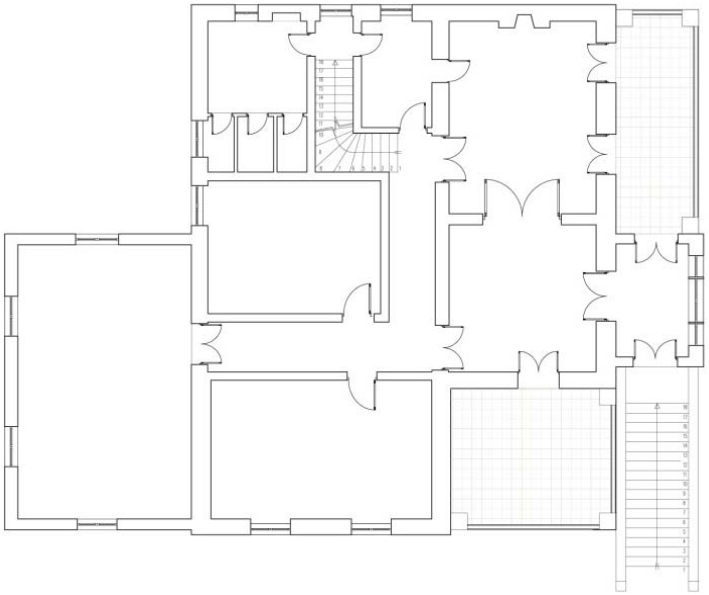


Figure 2. Ground floor plan of the building

3.1 NORMATIVE REFERENCES

The analysis and the safety verifications of the structure were performed in agreement with the following standards.

European standards:

- EN 1998-1:2005. Eurocode 8 - Design of structures for earthquake resistance. Part 1: General rules, seismic action and rules for buildings.
- EN 1998-3:2005. Eurocode 8 - Design of structures for earthquake resistance. Part 3: Strengthening and repair of buildings.

Italian standards:

- DM 14.1.2008. NTC2008 - Nuove norme tecniche per le costruzioni.
- Circolare 2.2.2009, n.617. Istruzioni per l'applicazione delle "Nuove norme tecniche per le costruzioni" di cui al D.M. 14.1.2008.
- DPCM 9.2.2011. Linee guida per la valutazione e la riduzione del rischio sismico del patrimonio culturale.

3.2 PERFORMANCE REQUIREMENTS

The performance requirements refer to the state of damage in the structure defined through three Limit State, namely Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL).

The safety evaluation of this building is performed with respect to the Limit State of Significant Damage (Limit State of Life Safeguard, according to the Italian standard). This Limit States is characterized as follows:

LS of Significant Damage (SD). The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.

For the selected Limit State the return period of the seismic action is equal to 475 years, corresponding to a probability of exceedance of 10% in 50 years.

3.3 SEISMIC ACTION

According to 3.2.2 (EN 1998-1), the earthquake motion at a given point on the surface is described by an elastic ground acceleration response spectrum. For the Limit State of

Significant Damage, the horizontal component of the seismic action was defined with the following parameters: *ground type A, design ground acceleration* $a_g = 0.24\ g$.

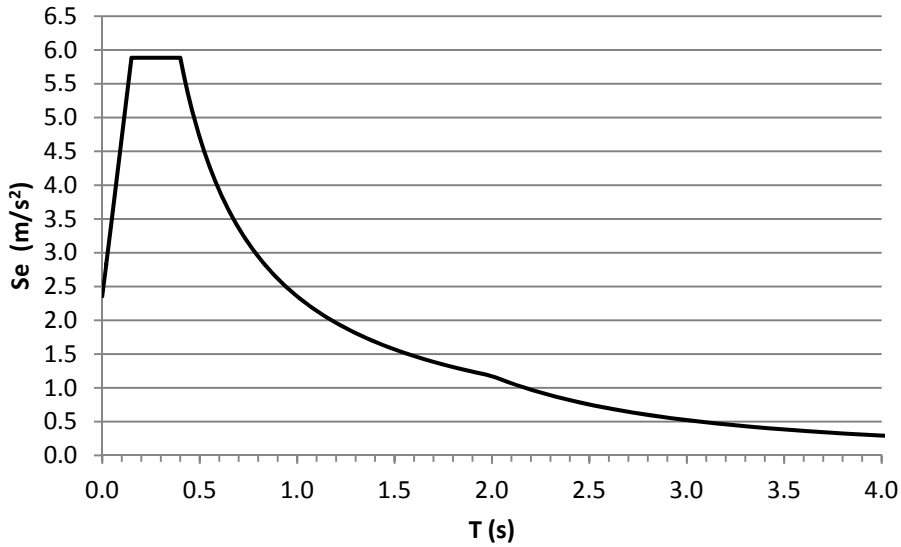


Figure 3. Horizontal design spectrum

The design seismic action is combined with the other appropriate permanent and variable actions in accordance with 3.2.4 (EN 1998-1).

3.4 KNOWLEDGE OF THE BUILDING

The knowledge of the existing masonry structure is vital for a proper analysis and it can be achieved with different depth levels depending on the accuracy of survey operations, historic analysis and experimental testing. The following three knowledge levels are defined:

- KL1: Limited knowledge
- KL2: Normal knowledge
- KL3: Full knowledge

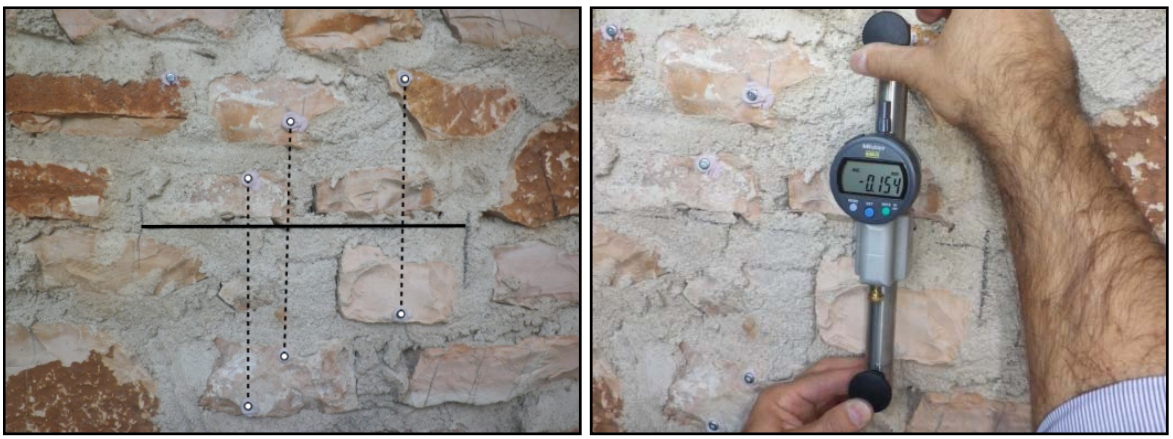
The factors which determine the appropriate knowledge level are: the geometrical properties of the structural system, the structural details and the mechanical properties of the constituent materials.

The overall structural geometry and member sizes are known from an extended survey, a procedure resulting in the production of structural drawings that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions.

The structural details are known from extended in-situ inspections. The following structural details were examined: quality of the connection between crossing walls, quality of the connection between floors and walls, presence of structurally efficient lintels above the openings, presence of structurally efficient elements able to counteract the thrusts that may be present, presence of elements, even non-structural, of high vulnerability, type of masonry (single-leaf, three-leaf, with or without transversal connections) and its characteristics (made of brick or stone, regular or irregular). In several points of the walls direct inspections were performed by removing the plaster in order to identify shape and size of the masonry elements. A photographic survey and drawings of the wall sections were produced.

The information on the mechanical properties of masonry are available from extended in-situ testing and from laboratory tests. Two single-flat-jack tests and a double-flat-jack test were performed in order to determine the local stress level and the local stress-strain behaviour of the masonry walls.

The single-flat-jack test is based on the release of the state of stress in a small area of the masonry by a plane cut perpendicular to the surface. The stress release allows the sides of the cut to close and the deformation (level of strain) can be assessed by measurements between a set of points symmetrically positioned on either side of the cut. The flat jack is then inserted into the cut and the pressure is gradually increased until the previously measured closure is nullified and the state of strain returns to the condition before the cut. In this condition the pressure in the jack is equal to the previously existing state of stress in the masonry provided that correction factors are applied. Figure 4 shows some step of the execution of the test.



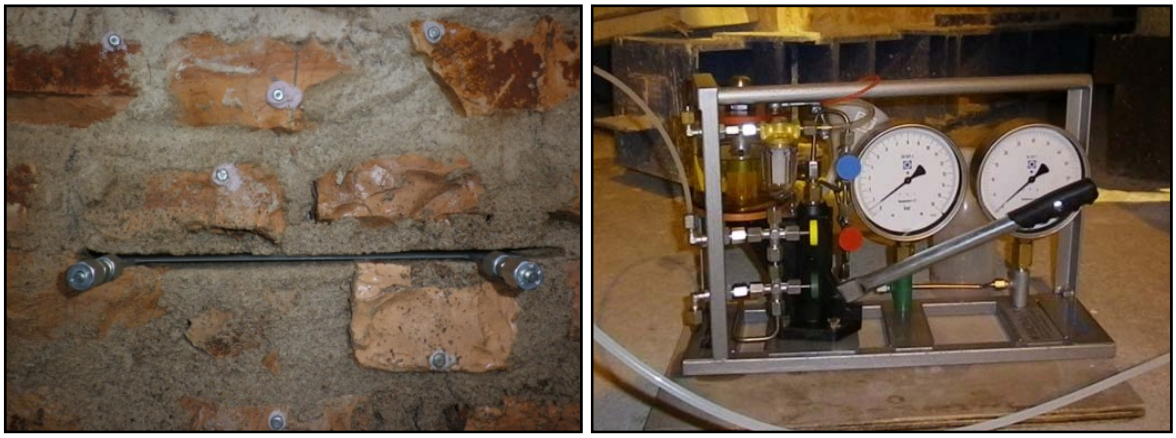


Figure 4. Single-flat-jack test

The double flat jack test is based on the use of two flat jacks on a common oil supply in order to apply a stress field to a volume of masonry between them. In this way it is possible to evaluate the local stress-strain behaviour of the masonry as well as its compressive strength. The test was performed, after the single flat jack test, inserting an additional flat jack above the one already put in place. Figure 5 shows the two flat jacks as well as the vertical and horizontal measuring positions while Figure 6 shows the stress-strain behaviour obtained from the test. More details are available in Chapter 4.

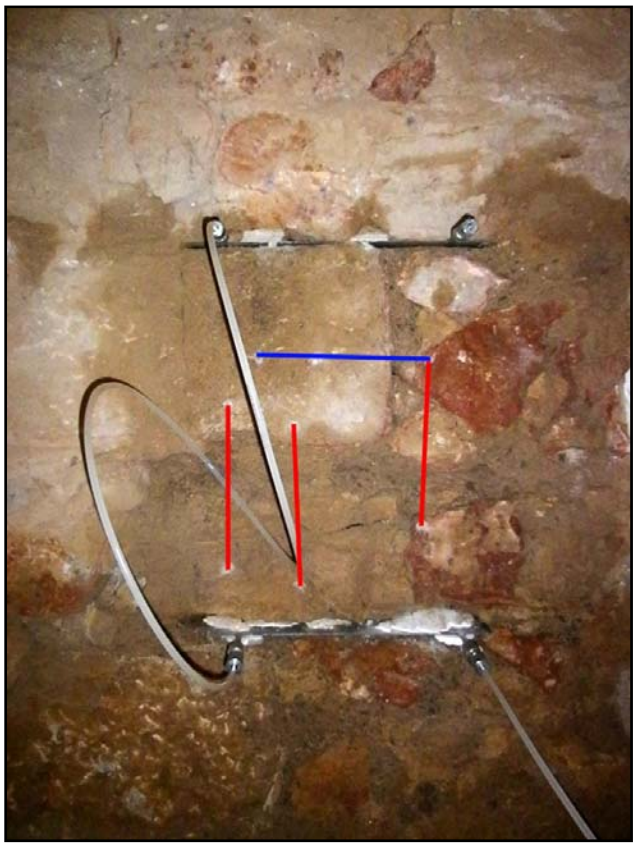


Figure 5. Single-flat-jack test

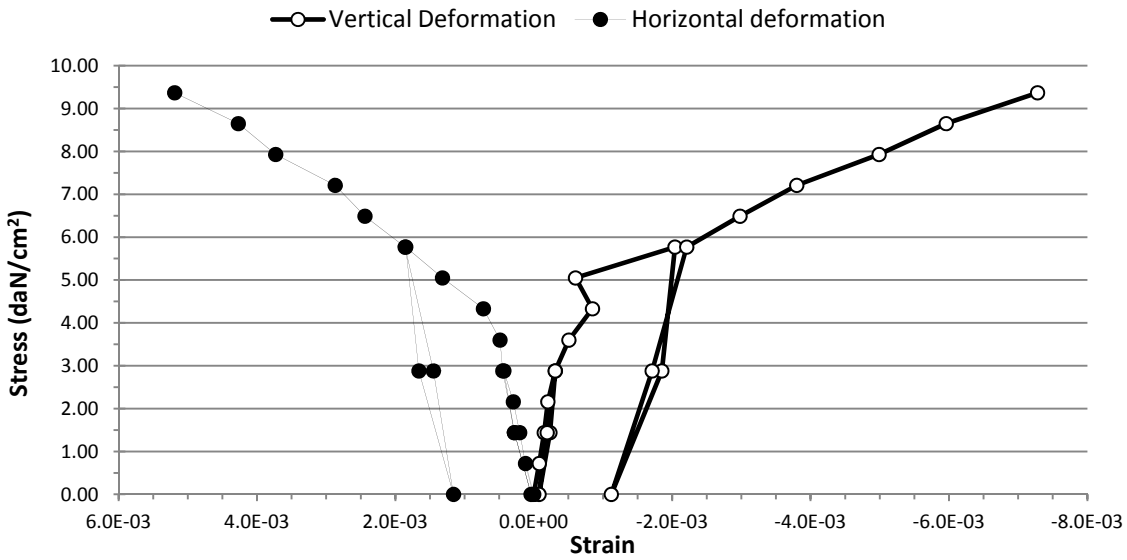


Figure 6. Stress-strain behavior

In addition to the flat jack tests, laboratory tests for material characterization were carried out on stones and bricks in order to identify their physical and mechanical properties.

According to the informations acquired on geometry, details and materials and with reference to Table 3.1 (EN 1998-3) or Table C8A.1.1 (Circolare 2.02.2009, n.617) the following knowledge level and the related confidence factor were selected:

- Knowledge level: KL2 Normal Knowledge
- Confidence factor: $CF_{KL2} = 1.2$

The confidence factor affects the mechanical characteristics of the materials both for linear and non-linear analysis. The results of the tests carried out on masonry in-situ and in laboratory were examined and considered as part of a general typological framework, taking into account results of other experimental tests available in literature for the wall types involved. This approach allows to evaluate, even statistically, the actual representativeness of the found values.

As far as regard masonry compressive strength, the results of the flat jack tests were compared with those calculated applying the following formula available in literature (Tassios-Chronopoulos):

$$f_{ex,c} = \frac{2/3 \cdot \sqrt{f_{bc}} + k_1 f_{mc} - k_2}{1 + 3.50(V_m/V_w - 0.30)}$$

where: $f_{ex,c}$ is the compressive strength of the external leaves, f_{bc} is the compressive strength of masonry elements, f_{mc} is the compressive strength of mortar, k_1 is a factor

describing the type of masonry (0.6 for rubble stones, 0.2 for bricks or regular stones), k_2 is a parameter describing the influence of blocks shape and the type of construction (0.0 for regular, 0.5 for semi-regular, 2.5 for rubble stone masonry), V_m/V_w is the ratio between volume of mortar and total volume.

Finally, according to C8A.1.A.4 (Circolare 2.02.2009, n.617) having achieved a knowledge level KL2, the mean values of the mechanical properties were defined by selecting the middle value of the ranges given in Table C8A.2.1 (Circolare 2.02.2009, n. 617) for the type of masonry constituting the walls. Such values are very close to those measured or calculated, and in any case more conservative. Table 1 shows the mechanical properties of masonry before and after the interventions (grouting).

Table 1. Mechanical properties of masonry

		Irregular stone masonry with weak internal core	**Irregular stone masonry consolidated with grout injection**	
Specific weight	**w**	19	19	kN/m^3
Modulus of Elasticity	**E**	653	1450	N/mm^2
Poisson ratio	**v**	0.40	0.40	
Compressive strength	$\mathbf{f_c}$	1.05	2.33	N/mm^2
Tensile strength	$\mathbf{f_t}$	0.10	0.20	N/mm^2
Initial shear strength	$\boldsymbol{\tau_0}$	0.10	0.20	N/mm^2

3.5 STRUCTURAL MODEL

The structure is modeled as an *equivalent frame* where each resistant wall is discretized by a set of masonry panels: *piers* and *spandrels*. The piers are vertical elements which provide resistance for both static and seismic loads; the spandrels are horizontal elements coupling the piers and representing the portion of wall above and below the openings. As shown in Figure 7, piers and spandrels are modeled as frame elements with shear deformation, while the joints are supposed infinitely resistant and stiff and are modeled by means of rigid offsets at the end of pier and spandrel elements. For in-plane behavior the length of the rigid offsets matches the intersection between piers and spandrels, while for out-of-plane behavior it generally corresponds to the thickness of the floors.

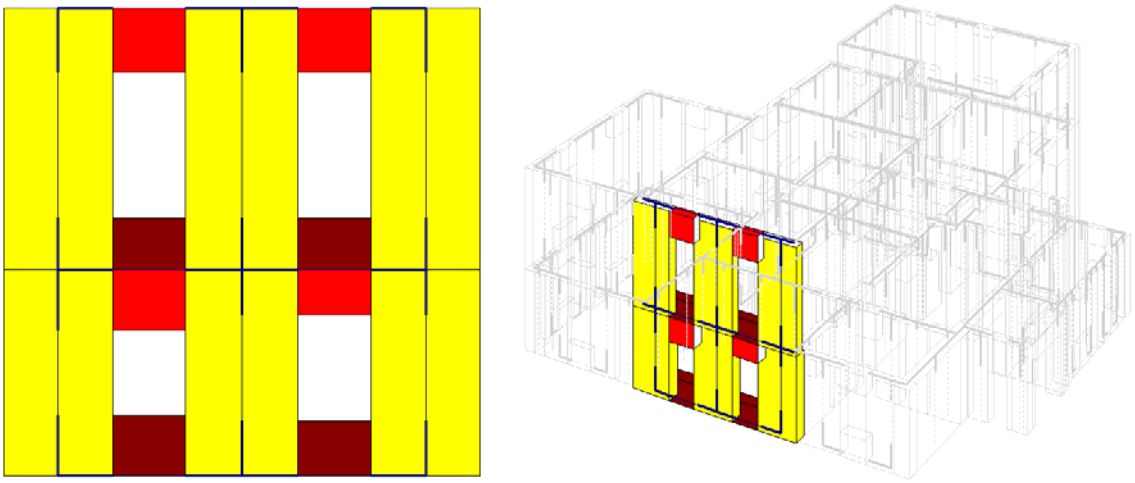


Figure 7. Equivalent frame model of a wall alignment

The model of the whole structure shown in Figure 8 consists of 413 frame elements and 307 nodes. It is obtained by assembling masonry walls and horizontal floors which were considered infinitely rigid in their plane. The nodes are free to rotate in the plane of the wall they belong to, while the nodes at the base are fully fixed.

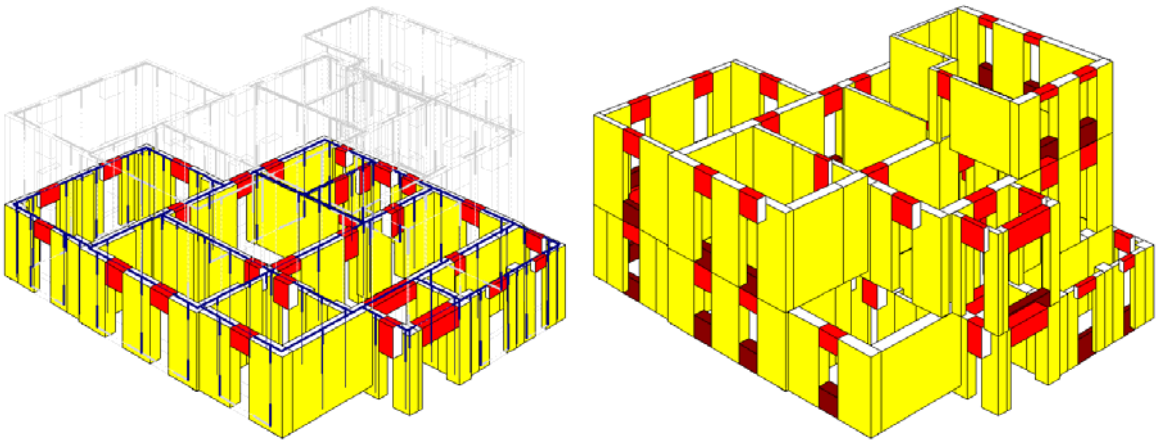


Figure 8. Equivalent frame model the whole structure

3.6 METHODS OF ANALYSIS

According to 4.4 (EN 1998-3:2005) and to 5.2 (DPCM 9.2.2011), the seismic action effects, combined with the effects of the other permanent and variable loads in accordance with the seismic load combination, may be evaluated using one of the following methods:

a) linear static analysis (lateral force);

b) linear dynamic analysis (modal response spectrum);

c) non-linear static analysis (pushover);

e* non-linear dynamic analysis (time history).!

Non-linear time history analysis cannot be considered a viable tool for practitioners and for the standard applications in residential buildings. Non-linear static analysis and linear analyses with the q-factor approach seem the real options for designers at the moment and they were performed in order to analyze the structure at the initial state and after the strengthening interventions, consolidation of the masonry walls by mean of grout injections.

The structural model of the building and all the analysis were executed by mean of the software Aedes PC.E.

3.7 LINEAR STATIC ANALYSIS

In linear static analysis the reference seismic action for the ultimate limit state is reduced by a behavior factor (q) to allow for a verification in the elastic range. In this way the analysis account for the additional displacement capacity of the structure when it reaches the maximum resistance and before the ultimate limit state. The application of this method in case of historic buildings can be problematic given the difficulties in defining a proper behavior factor with possible implications on the definition of the interventions.

In this case the analysis was performed as described in 4.3.3.2 (EN 1998-1) and in 7.3.3.2 (NTC2008) along the positive X direction using the behaviour factor q = 1.5. The seismic base shear force F_b was determined using the following expression:

$$F_b = S_d(T_1) \cdot m \cdot \lambda$$

where: $S_d(T_1)$ is the ordinate of the design spectrum, T_1 is the fundamental period of vibration in the direction considered, m is the total mass of the building and λ is a correction factor equal to 1 for building irregular in elevation.

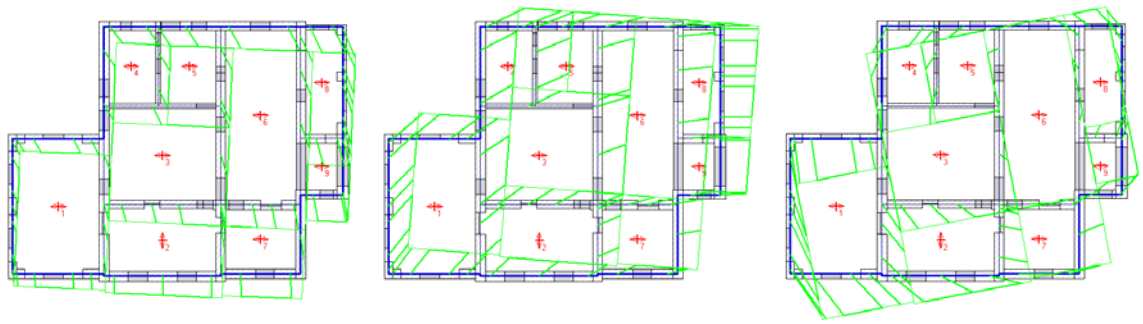


Figure 9. Modal deformed shape of the first three modes

A modal analysis has been performed in order to evaluate the fundamental period of vibration. Table 3 shows the first three modes of the structure, for each mode the period and the participating mass ratio are listed.

Table 3. Modal properties of the structure

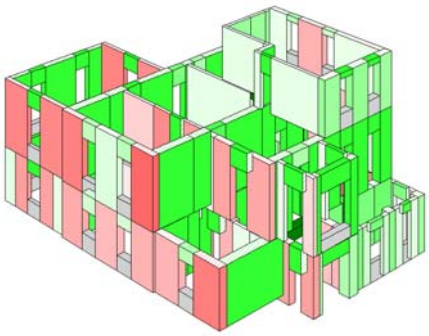
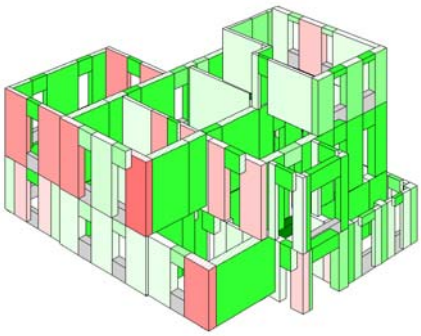
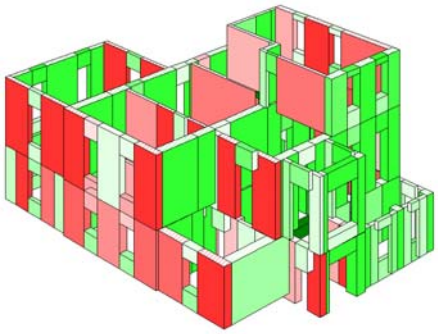
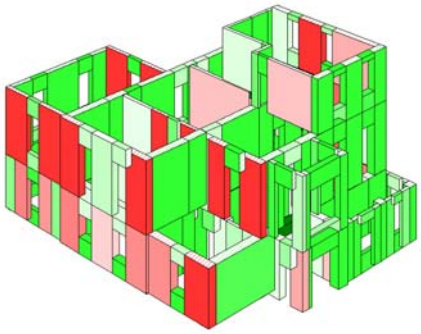
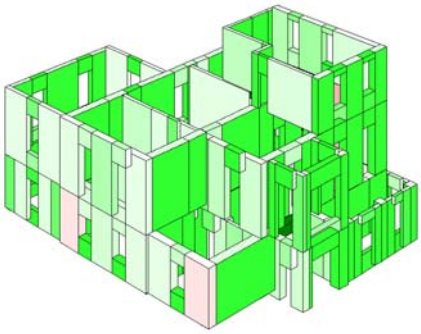
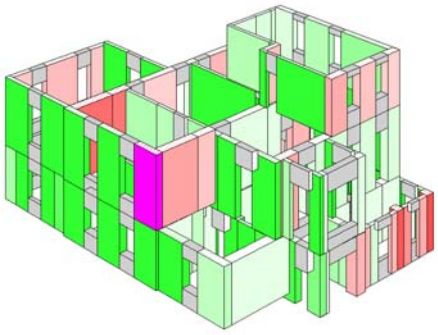
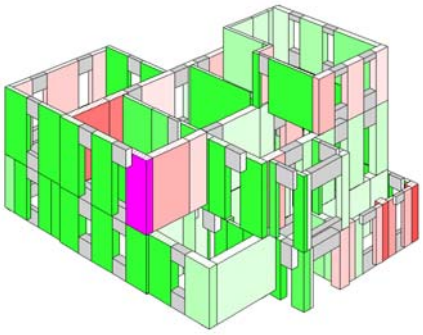
	Mode	Period (s)	Ux	Uy
At initial state	1	0.186	0.075	0.675
	2	0.180	0.717	0.118
	3	0.150	0.082	0.066
After interventions	1	0.125	0.075	0.676
	2	0.111	0.717	0.118
	3	0.101	0.082	0.066

The safety verifications in terms of global seismic behavior were executed according to C8.7.1.4 (Circolare 2.2.2099, n.617). The verifications are carried out in terms of strength: for each seismic resistant structural element the capacity must be greater than the seismic demand with respect to each of the following failure mode:

- in-plane flexure (7.2.2.1 - NTC2008)
- sliding shear (7.2.2.2 - NTC2008)
- diagonal cracking shear (C8.7.1.5 - Circolare 2.2.2099, n.617)
- out-of-plane flexure (7.2.2.3 - NTC2008)

The following table presents the results of the safety verifications proposing a comparison between the building at its initial state and after the interventions of grouting. In the following images the red elements are those for which the check is not satisfied and their amount is shown below.

The comparison shows a substantial improvement in the behaviour of the structure after the interventions. We can notice a large reduction of the elements that do not satisfy the checks. However, the consolidation of the masonry walls appear not to be sufficient for the structure to fulfil the requirements and the structure need further interventions.

At initial state	After interventions
In-plane flexure	
	
41 frame elements	*20 frame elements*
Sliding shear	
	
53 frame elements	*31 frame elements*
Diagonal cracking shear	
	
35 frame elements	*8 frame elements*
Out-of-plane flexure	
	
25 frame elements	*23 frame elements*

3.8 NON-LINEAR STATIC ANALYSIS

The pushover analysis was performed according to 4.3.3.4 (EN 1998-1) and to 7.3.4 (NTC2008) with the following algorithm:

1. The analysis is performed along a given direction and for a given distribution of the lateral loads. A proper base shear increment is chosen, suggested values range from 1/50 to 1/10 of the maximum base shear.
2. The structure subjected only to vertical loads is analyzed.
3. The base shear increment is applied, the lateral forces are distributed to the structure according to the chosen load pattern.
4. The internal actions of the structural elements under the combination of vertical and lateral loads are calculated. At each step of the analysis the incremental internal actions and displacements are added to the corresponding values of the previous step.
5. The total base shear and the control displacement are calculated, their values represent one of the points of the capacity curve.
6. The structural elements are subjected to the following verifications: in-plane flexure, sliding shear, diagonal cracking shear. If all the verifications are satisfied, the internal constraints of the element remain the same. When one of the verifications is not any more satisfied the structure enters the plastic range and plastic hinges have to be defined in the model to account for the new behavior of the element which develops increasing deformation under constant internal actions. In the sections of the elements where the verification is performed, if the shear reaches the maximum value, it has to remain constant in the next incremental steps: the secant shear stiffness will progressively decrease while the tangent shear stiffness becomes null. In order to account for this behavior the element is turned into a truss element. In this way in the next steps of the analysis the shear acting in the frame will not increase. The verifications in terms of axial stress will still be performed checking if the variation of the axial force leads to an exceedance of compressive or tensile strength. If the verification in terms of in-plane flexure is not satisfied at one of the end sections of the element, a plastic hinge is defined at that point for the next steps. Even in this case while the tangent rotational stiffness becomes null, the secant stiffness progressively decrease. After the insertion of the plastic hinge the incremental bending moment become null and the total bending moment in the given section remain constant. If one or more verifications are not satisfied the structural model has to be revised and the stiffness matrix has to be updated according to the new internal constraints. In case of rigid offsets at the ends of the

frame, the plastic hinges are defined at the ends of the deformable part. If the distribution of the lateral loads is proportional to the modal shape and adaptive, that is it follows the dynamic characteristics of the structure, the pattern must be updated whenever the model is revised. In other words, the changes in the structural model lead to new modes of vibration and therefore to a new distribution of lateral loads.

7. The steps 3, 4, 5, 6 are repeated until one of the piers reaches a collapse ultimate state: excessive in-plane deformation (the limit is defined as a percentage of the pier deformable height: 0.4% if the first plastic hinge occurs for shear, 0.6% if it occurs for flexure); the element is nonreactive because of tensile deformation; the element reached the maximum resistance in terms of out-of-plane flexure. In this way a capacity curve like the one shown in Figure 10a is drawn.
8. When one or more piers reach the collapse ultimate state there is a drop in terms of resistance: the incremental procedure cannot continue and it is necessary to repeat the all procedure starting from step 2 using a structural model that account for the elements that entered the plastic range and for the ones that reached collapse. In this way several sub-curves are obtained like the ones shown in Figure 10b.
9. The procedure stops when the structure becomes a mechanism or for an excessive value of the control displacement.
10. The final capacity curve that account for the drops in resistance of the structure is obtained connecting the sub-curves with vertical segments as in Figure 10c.

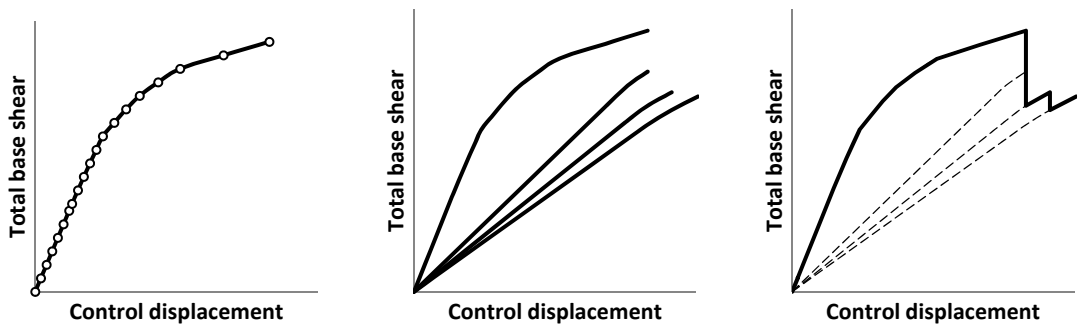


Figure 10. Capacity curves

In this case the analysis was performed along the positive X direction and a “triangular” distribution of the lateral loads was applied: the pattern is based on lateral forces that are proportional to mass and elevation, as the one used for static linear analysis. Since the structure at the last story of the building is rather small if compared with the other stories, the control displacement was chosen at the level of the second floor.

Figure 11 shows the capacity curves, relation between base shear force and control displacement, both at the initial state of the building and after the interventions. The dashed line represent the target displacement, defined as the seismic demand derived from the elastic response spectrum in terms of the displacement of an equivalent single-degree-of-freedom system.

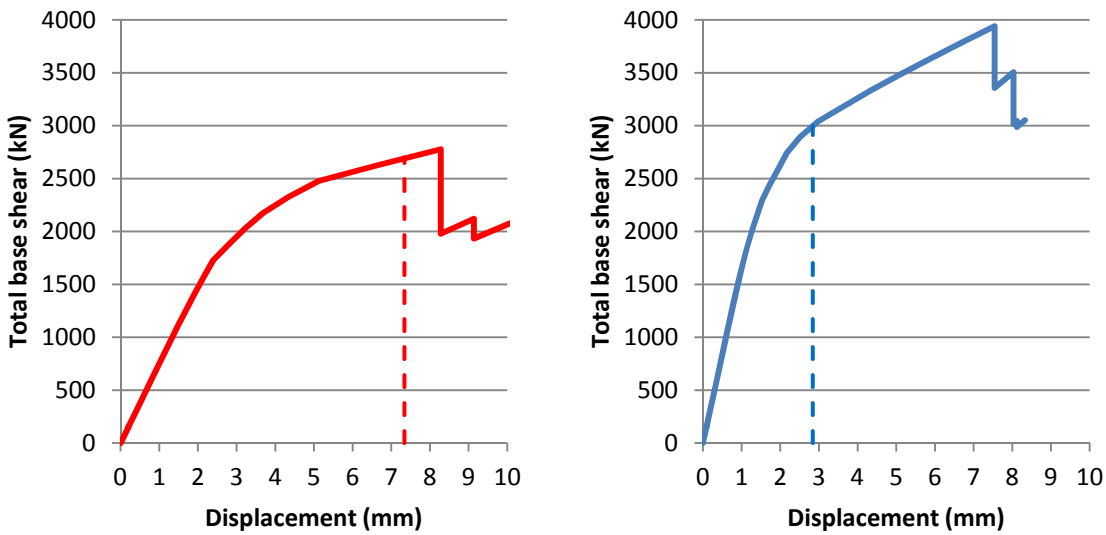


Figure 11. Capacity curves: a) at initial state, b) after interventions.

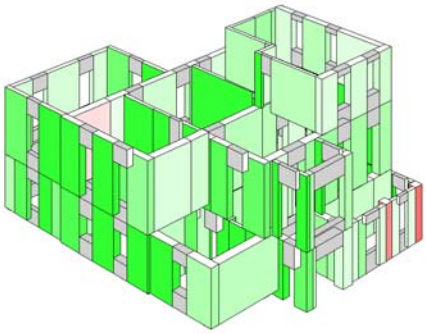
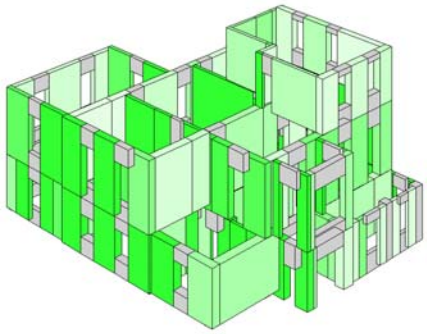
Both curves satisfy the check: the displacement at the ultimate limit state is greater than the required. However, the capacity curve which refers to the initial state of the building intersects the required displacement quite late, for that displacement a big amount of elements have entered the plastic range and the structure has suffered heavy damages. On the other hand the capacity curve after the interventions shows clearly an increase in the stiffness of the structures and intersects the required displacement just at the begin of the plastic range.

The pushover analysis allows to calculate the real overstrength ratio α_u/α_1 of the structure, where α_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, α_u is the value by which the horizontal seismic design action is multiplied in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability.

According to C8.7.1.2 (Circolare 2.2.2009, n.617) for building that are irregular in elevation the behavior factor can be calculated as follow:

$$q = 1.5 \cdot \alpha_u/\alpha_1$$

Pushover analysis aims to assess the in-plane behavior of the structure. In order to evaluate the resistance with regards to out-of-plane behavior, a response spectrum analysis was performed using the behavior factor obtained from the pushover analysis. The following table shows the values of the overstrength ratio and the behavior factor for the analyses at initial state and after interventions along with the results of the safety verifications in terms of out-of-plane flexure and number of frame that do not satisfy the check.

At initial state	**After interventions**
$\alpha_u/\alpha_1 = 1.957$ $q = 2.935$	$\alpha_u/\alpha_1 = 2.303$ $q = 3.454$
Out-of-plane flexure	
	
3 frame elements	*0 frame elements*

The comparison between the situation at the initial state and after interventions proved the efficiency of the interventions. After strengthening the structures satisfies the verifications both for in-plane and out-of-plane behavior.

3.9 CONLUSIONS

The structure was analyzed by mean of linear and non-linear analyses both before and after the strengthening interventions. Linear static analysis showed an improvement in the behavior of the structure but not sufficient to meet the requirements of the standards. On the other non-linear pushover analysis proved that with same interventions the structure fulfill the requirements.

The results of linear analysis appear to be more conservative but, as a general remark, when dealing with existing buildings, an excessively conservative underestimation of the global seismic resistance is to be avoided. While the strengthening interventions that aim to increase the quality of the connections in order to avoid out-of-plane mechanism are in general always positive, the increase in global strength is more difficult to achieve and it is pursued sometimes with heavy, non conservative (from the architectural point of view)

interventions whose effects may be very difficult to predict, and whose effectiveness can be sometimes arguable, as past experiences have shown.

4

Flat Jack Test

The application of the flat jack test for the detection of the state of stress in compression and the stress-strain behaviour of historic masonry buildings was introduced in 1978.

The flat jack test allows to determine the local stress level (single flat jack) and the local stress-strain behaviour (double flat jack).

For the building analyzed in chapter 3, the following tests have been performed:

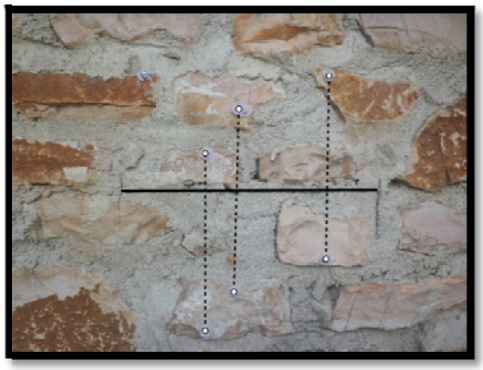
- 2 single flat jack tests
- 1 double flat jack test

4.1 DETERMINATION OF LOCAL STRESS ON SITE (SINGLE FLAT JACK)

The testing technique is based on the release of the state of stress in a small area of the masonry by a plane cut perpendicular to the surface. The stress release allows the sides of the cut to close and the extent (level of strain) can be assessed by measurements between two points symmetrically positioned on either side of the cut. The flat jack is then inserted into the cut and the pressure is gradually increased until the previously measured closure is nullified and the state of strain is returned to the condition prevailing before the cut. In this condition the pressure in the jack is equal to the previously existing state of stress in the masonry provided that correction factors are applied.

The test sequence is as follow:

1. Choose a representative piece of masonry and glue the metal reference points on either side of the selected cut line. Where possible, the cut for insertion of the jack should be made into a layer of mortar.

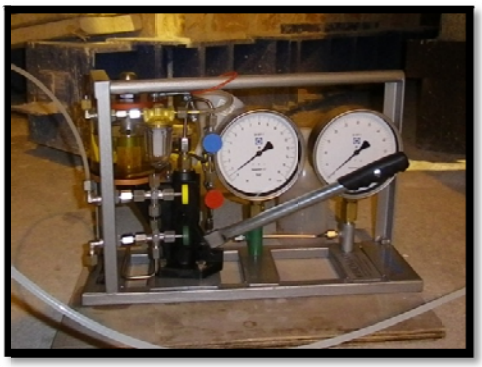


2. When the glue has set, a set of initial reference measurements are taken with the removable strain gauge.

3. The cut is then made taking care to disturb the surrounding area as little as possible.
4. After cutting and cleaning, a second set of measurements are taken with the removable strain gauge in order to assess the closure of the cut.

5. The jack is then inserted into the cut.
6. Purging of the jack to eliminate any air bubbles in the circuit.

7. Increase the pressure in the jack in increments no less than 0.5 bar and monitor the strain after a short dwell at each increment.
8. The test is stopped when the strain is returned to the state measured before the cut was made with no single deviation exceeding 10% for all measurements.



The restoring stress value σ_v at the tested point is given by the relation:

$$\sigma_v = p \cdot \frac{A_j}{A_{slot}} \cdot k_m = p \cdot k_t \cdot k_m$$

where: p is the pressure which restores the original strain condition; A_j is the area of the flat jack; A_{slot} is the area of the slot; k_m is a dimensionless efficiency constant which takes into account the geometrical characteristics of the jack; $k_t = A_j / A_{slot}$ is a dimensionless geometrical constant which takes into account the relative size of the jack and the slot.

4.2 DETERMINATION OF STRESS-STRAIN BEHAVIOUR (DOUBLE FLAT JACK)

The testing technique is based on the use of two flat jacks on a common oil supply to apply a stress field to a volume of masonry between them. In this way it is possible to evaluate the local stress-strain behaviour of the masonry as well as its compressive strength.

The test may be performed, after the single flat jack test, by either inserting an additional flat jack above or below the one already put in place or without the prior measurement of the stress state.

The test sequence is as follows:

1. Choose a representative piece of masonry, then glue the metal reference points at the correct gauge length for the strain measuring instrument. Several measuring positions should be used and their results averaged.
2. The cuts are then made taking care to disturb the surrounding area as little as possible. Slots shall be parallel, vertically aligned and at a distance of about 1.5 times the length of the flat jacks.
3. The jacks are then inserted into the cut slots.
4. After the zero strain measurement has been taken, the pressure is then increased in increments of about 10% or less of the expected maximum and the strain is monitored after a short dwell at each increment. Both jack pressure and strain should be recorded at each increment . The ratio of the increase of jack pressure (dp) to the strain increment (de_m) should be monitored and the test should be stopped when the ratio starts dropping rapidly to avoid damage to the masonry.
5. Depressurise and remove the jacks.

The stress in the masonry between the jacks σ_v is given by the relation:

$$\sigma_v = p \cdot \frac{A_j}{\bar{A}_{slot}} \cdot \bar{k}_m = p \cdot \bar{k}_t \cdot \bar{k}_m$$

where: p is the hydraulic pressure in the jack lines; A_j is the area of the flat jack; $\bar{A}_{slot}$ is the average area of the two slots; k_m is the mean value of the dimensionless efficiency

constants of the two jacks; $k_t = A_j / \bar{A}_{slot}$ is the mean value of the dimensionless geometrical constants of the two jacks.

4.3 DESCRIPTION OF THE EQUIPMENT USED

The following equipment has been used:

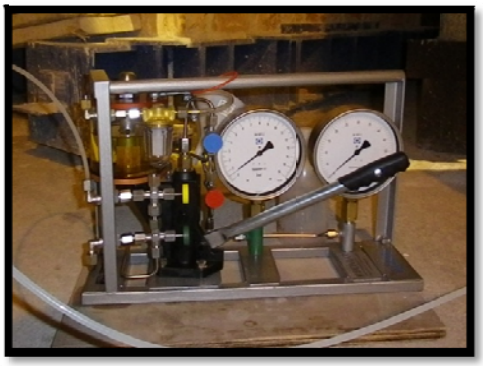
1. The slot cutting equipment is a hydraulic cutter type Husqwarna K960 Ring with diamond blade. It provides a precise cut of minimum disturbance to the structure. The cut, 4 mm in height, has a shape identical to the one of the flat jack.

2. The strain measuring equipment is a removable deformometer type DGEI250, with the following characteristics:
   Gauge length: 250mm
   Range of use: 10mm (±5mm)
   Resolution: 0.001 mm

3. The flat jacks used have elongated semi-circular shape with the following characteristics:

model:	Boviar MP-8A
dimensions:	350 x 259 x 4.2 mm
area:	77506 mm^2
sheets thickness:	0.8 mm

4. The hydraulic pump is a hand pump for oil operation type Glötzl M2H16.

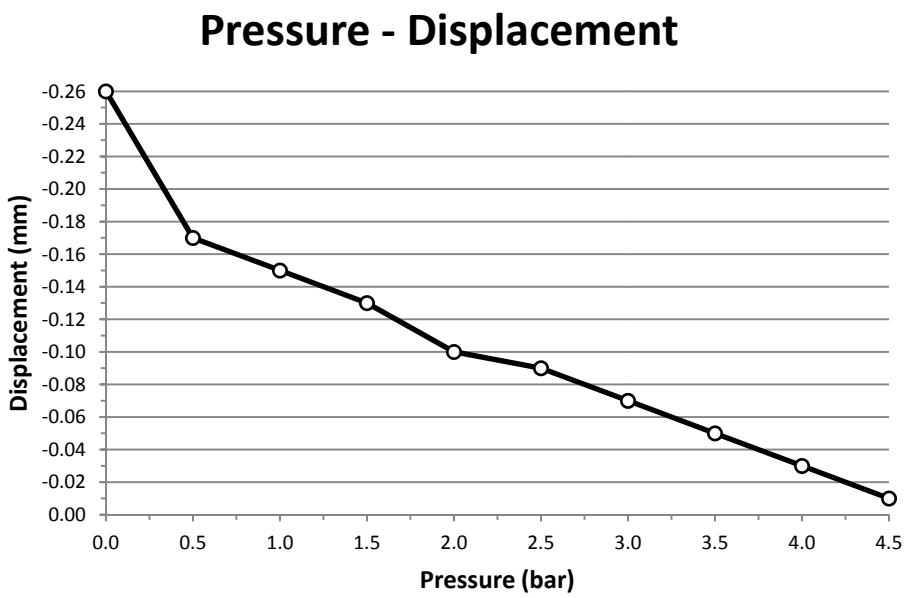


4.4 TEST RESULTS

Three tests were performed on two three-leaf masonry walls at the ground floor of the building: a single and a double flat jack test on the internal facade of an external wall, as well as a single flat jack test on an internal wall.

4.4.1 Test 1

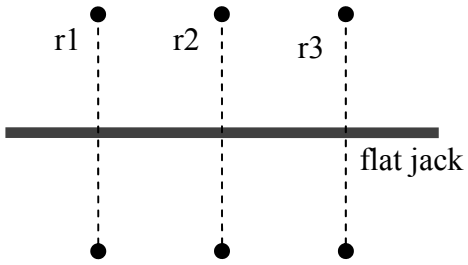
Test 1 was a single flat jack test performed on the internal façade of an external wall that has been chosen as the one that carry the highest load. The wall is a three-leaf masonry with stones of large dimension.



Test 1

Position	Internal façade of an external wall
Material of the wall	Three-leaf masonry
Type of test	Single flat jack

Gauge length	L	=	200	mm
Gauge constant	G_k	=	0.8	
Flat jack area	A_j	=	77506	mm^2
Slot area	A_{slot}	=	86000	mm^2
Efficiency coefficient	k_m	=	0.8	
Geometric coefficient	$k_t = A_j / A_{slot}$	=	0.9	



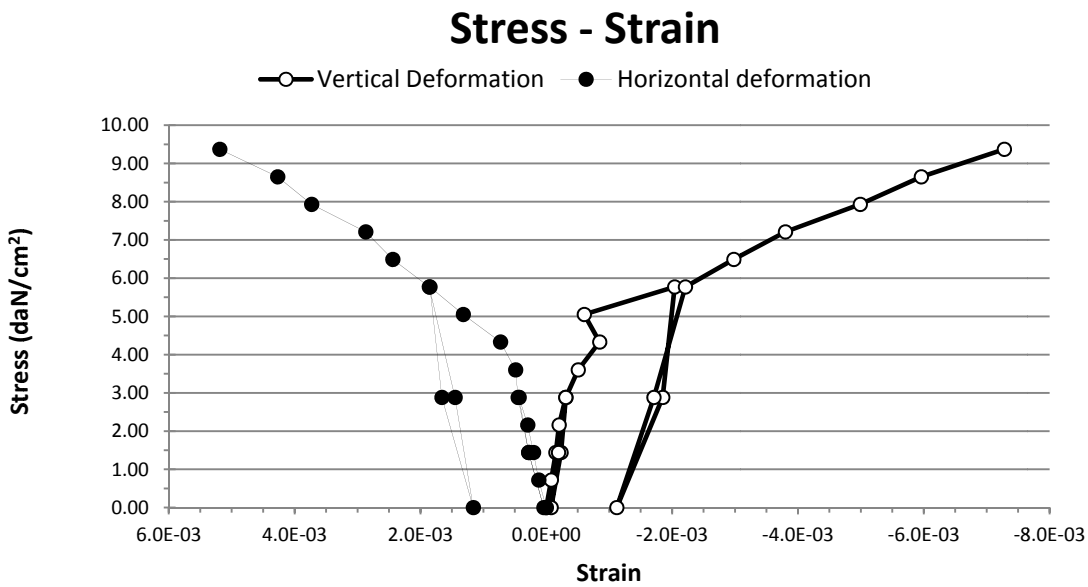
Reference measurements	r1 mm	r2 mm	r3 mm	Average mm
before cutting	-0.749	-0.059	-0.270	-0.359 (A_1)
after cutting	-0.407	0.363	-0.070	-0.038

Pressure bar	Stress daN/cm^2	r1 mm	r2 mm	r3 mm	Average (A_2) mm	Displacement = $(A_1-A_2)\cdot G_k$ mm
0.00	0.00	-0.407	0.363	-0.070	-0.038	-0.26
0.50	0.36	-0.519	0.172	-0.078	-0.142	-0.17
1.00	0.72	-0.556	0.140	-0.096	-0.171	-0.15
1.50	1.08	-0.583	0.106	-0.124	-0.200	-0.13
2.00	1.44	-0.615	0.065	-0.143	-0.231	-0.10
2.50	1.80	-0.624	0.040	-0.162	-0.249	-0.09
3.00	2.16	-0.659	0.018	-0.185	-0.275	-0.07
3.50	2.52	-0.680	-0.018	-0.210	-0.303	-0.05
4.00	2.88	-0.693	-0.029	-0.233	-0.318	-0.03
4.50	3.24	-0.712	-0.050	-0.265	-0.342	-0.01

Restoring Pressure	4.35	bar
Restoring Stress	3.13	daN/cm^2

4.4.2 Test 2

A double flat jack test was performed to determine the local stress-strain behavior in the wall tested in Test 1.

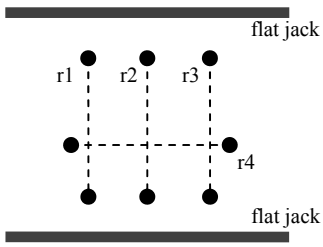


- a vertical crack was observed at the pressure of 8 bar;
- a crack opened horizontally at a joint at the pressure of 13 bar;
- between the 8 and 13 bar pressure, noises due to cracking were heard and small pieces of mortar were falling.

Test 2

Position	Internal façade of an external wall
Material of the wall	Three-leaf masonry
Type of test	Double flat jack

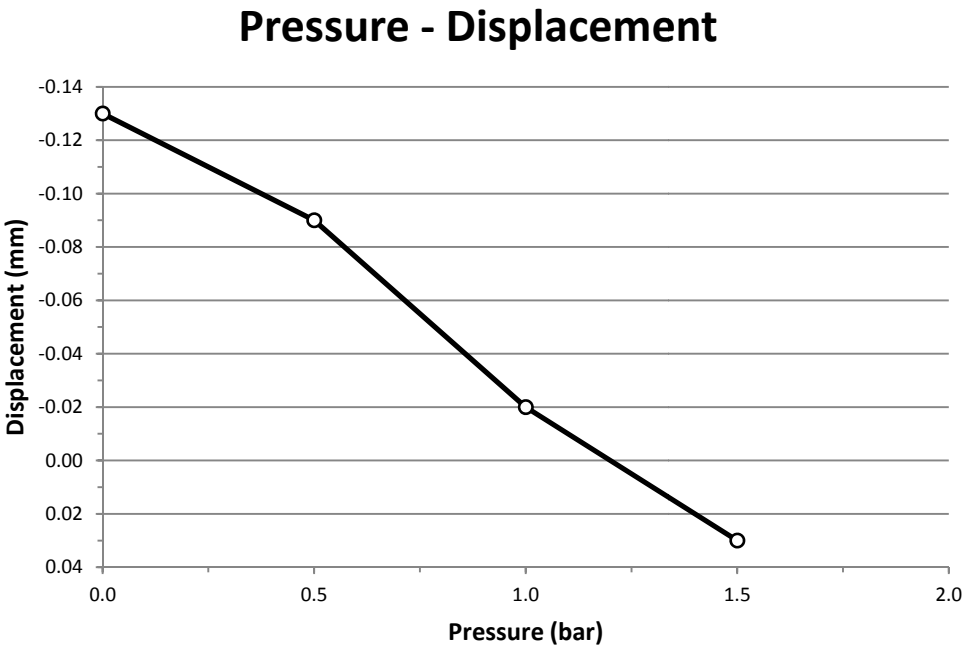
Gauge length	L	=	200	mm
Gauge constant	G_k	=	0.8	
Flat jack area	A_j	=	77506	mm^2
Slot area	A_{slot}	=	86000	mm^2
Efficiency coefficient	k_m	=	0.8	
Geometric coefficient	$k_t = A_j / A_{slot}$	=	0.9	



Pressure bar	Stress daN/cm^2	r1 mm	r2 mm	r3 mm	Mean-V mm	Strain-V mm/mm	r4 mm	Strain-O mm/mm
0.00	0.00	-0.204	-0.221	-0.241	-0.222	0.00E+0	-0.978	0.00E+0
1.00	0.72	-0.195	-0.193	-0.219	-0.202	-7.87E-5	-1.008	1.20E-4
2.00	1.44	-0.180	-0.177	-0.196	-0.184	-1.51E-4	-1.029	2.04E-4
3.00	2.16	-0.169	-0.157	-0.188	-0.171	-2.03E-4	-1.052	2.96E-4
4.00	2.88	-0.156	-0.131	-0.146	-0.144	-3.11E-4	-1.086	4.32E-4
2.00	1.44	-0.170	-0.159	-0.170	-0.166	-2.33E-4	-1.048	2.80E-4
0.00	0.00	-0.205	-0.208	-0.196	-0.203	-7.60E-5	-0.988	4.00E-5
2.00	1.44	-0.183	-0.166	-0.172	-0.174	-1.93E-4	-1.048	2.80E-4
4.00	2.88	-0.158	-0.132	-0.144	-0.145	-3.09E-4	-1.090	4.48E-4
5.00	3.60	-0.136	-0.102	-0.048	-0.095	-5.07E-4	-1.100	4.88E-4
6.00	4.33	-0.110	-0.057	0.137	-0.010	-8.48E-4	-1.160	7.28E-4
7.00	5.05	-0.057	-0.032	-0.125	-0.071	-6.03E-4	-1.308	1.32E-3
8.00	5.77	0.027	0.172	0.662	0.287	-2.04E-3	-1.440	1.85E-3
4.00	2.88	-0.007	110	0.618	0.240	-1.85E-3	-1.394	1.66E-3
0.00	0.00	-0.144	-0.095	0.416	0.059	-1.12E-3	-1.269	1.16E-3
4.00	2.88	-0.040	0.080	0.575	0.205	-1.71E-3	-1.340	1.45E-3
8.00	5.77	0.037	0.203	0.755	0.332	-2.21E-3	-1.443	1.86E-3
9.00	6.49	0.141	0.358	1.067	0.522	-2.98E-3	-1.587	2.44E-3
10.00	7.21	0.245	0.540	1.397	0.727	-3.80E-3	-1.695	2.87E-3
11.00	7.93	0.417	0.801	1.856	1.025	-4.99E-3	-1.911	3.73E-3
12.00	8.65	0.575	1.021	2.205	1.267	-5.96E-3	-2.045	4.27E-3
13.00	9.37	0.745	1.320	2.730	1.598	-7.28E-3	-2275	5.19E-3

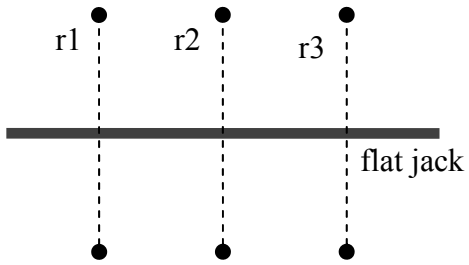
4.4.3 Test 3

A single flat jack test was performed on an internal wall made up of three-leaf masonry. The stones which constitute the masonry were of small dimension and the slot obtained with the cut was quite irregular. Nevertheless, it was possible to perform the test.



Test 3				
Position	Internal wall			
Material of the wall	Three-leaf masonry			
Type of test	Single flat jack			

Gauge length	L	=	200	mm
Gauge constant	G_k	=	0.8	
Flat jack area	A_j	=	77506	mm^2
Slot area	A_{slot}	=	86000	mm^2
Efficiency coefficient	k_m	=	0.8	
Geometric coefficient	$k_t = A_j / A_{slot}$	=	0.9	



Reference measurements	r1 mm	r2 mm	r3 mm	Average mm
before cutting	-0.026	-0.026	0.172	0.040
after cutting	0.151	0.151	0.307	0.203

Pressure bar	Stress daN/cm2	r1 mm	r2 mm	r3 mm	Average mm	Displacement =(A1-A2)·Gk mm
0.00	0.00	0.151	0.151	0.307	0.203	-0.13
0.50	0.36	0.097	0.097	0.263	0.152	-0.09
1.00	0.72	-0.002	-0.002	0.192	0.063	-0.02
1.50	1.08	-0.064	-0.064	0.141	0.004	0.03

Restoring Pressure	1.22	bar
Restoring Stress	0.88	daN/cm^2

5

Local seismic analysis of the church of Sts Helen and Constantine in Piraeus

The Church of St. Constantine & Helen is located in the Municipal Theater square in Piraeus. It was constructed in 1882. A series of recent earthquakes, including the Athens 1999 earthquake, strained the temple and caused serious damages. These damages are mainly attributed to the absence of particular provisions to carry the earthquake loads. In order to evaluate the earthquake vulnerability of the church, a detailed study of the masonry structure was conducted in order to determine the mechanical properties, the building construction details and the current condition of the structure as well as its dynamic behavior through in-situ and laboratory testing as well as through finite element analysis. Based on the results, strengthening and repair measures were proposed, using a combination of masonry consolidation techniques, fiber reinforced plastic materials (FRP) and steel tie rods. [15]

This chapter deals with a local non-linear analysis of the church in order to evaluate its transversal seismic response. The analysis of the damages the churches have suffered after major earthquakes in recent decades revealed that this kind of structures can be analyzed as a set of architectonic portions characterized by a structural response basically independent from the church as a whole: the macroelements. The methods used for the structural modeling and the results of the analyses will be presented.

5.1 DESCRIPTION OF THE STRUCTURE

The Church is considered as one of the most majestic temples of Piraeus and was based on the blueprints of the architect John Lazarimos. Moreover, it is characterized by a capacity of 1200 people, its impressive bell-towers and a high dome. After the end of the construction, many prominent hagiographers and marble-sculptors undertook the decoration of the church interior.

From an architectural perspective, the church is a “domed basilica”. Its dome is carried on four concave triangular pendentives that serve to the transition from the circular base of the dome to its rectangular base. The weight of the dome passes through the pendentives to four massive piers at the corners. The length of the temple is 29m and the width is 23m. The total height of the church is 23.4m with the dome and the bell towers being 7.2m and 8.6m high, respectively.

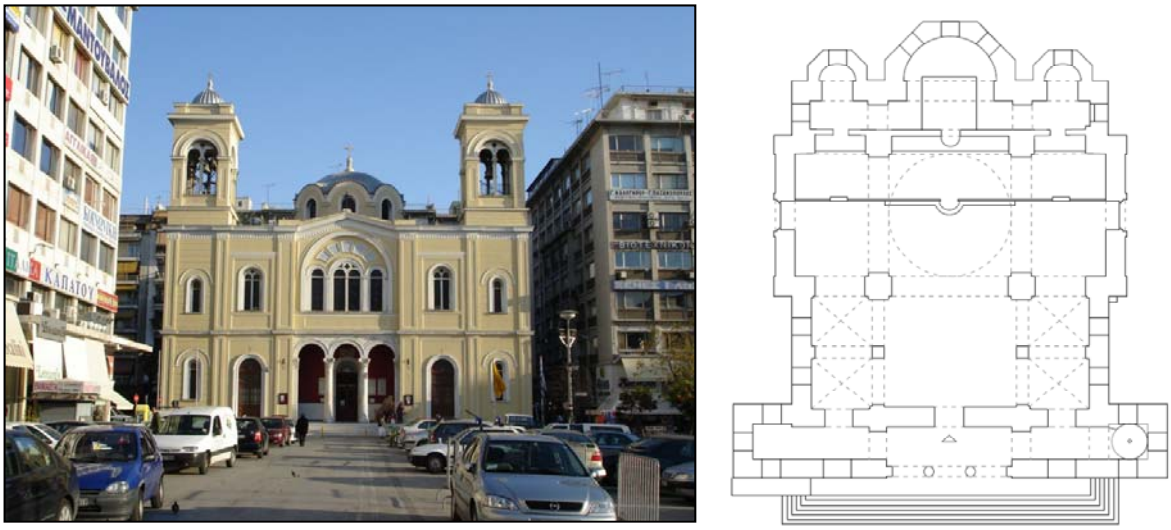


Figure 1. Front view and plan of the church

5.2 MECHANICAL PROPERTIES OF MASONRY

On the base of extended laboratory and in-situ testing carried out on the structural elements of the church [15], the following value of the compressive strength of the masonry elements (stones and mortars) were defined:

Table 1. Compression strength of stones and mortar

		Stones	Mortar	
Compressive strength	f_c	30	1	N/mm^2

At the same time, it was possible to calculate the mechanical properties of the masonry that constitutes the walls and the columns, both before and after the interventions of grout injection:

Table 2. Mechanical properties of masonry

		At initial state	After interventions	
Specific weight	**w**	22	22	kN/m^3
Modulus of Elasticity	**E**	3150	4710	N/mm^2
Shear Modulus	**G**	1358	2030	N/mm^2
Compressive strength	f_c	3.15	4.71	N/mm^2
Tensile strength	f_t	0.31	0.47	N/mm^2

5.3 GLOBAL AND LOCAL ANALYSES

The analysis of the damages the churches have suffered after major earthquakes in recent decades revealed that this kind of structures can be analyzed as a set of architectonic portions characterized by a structural response basically independent from the church as a whole. These portions, referred to as macroelements, can be identified as the façade, the apse, the bell tower, the dome, the triumphal arch etc..

For each macroelement a linear or non-linear static analysis can be performed by means of a finite element model. Another option is the kinematic limit analysis: the uncertainties in choosing a priori the collapse mechanism, which is the critical point of the kinematic approach in limit analysis of structures, are in this case very small thanks to the in-depth knowledge of the church behavior resulting from an accurate survey of the damages.

In addition to the state of damage, the systematic analysis of the construction details, such as the quality of masonry or the presence of earthquake-resistant elements, can help to individuate the macroelements and to assess which collapse mechanism is more likely to develop.

The necessity of a global analysis extended to the whole structure or a local analysis on the single macroelement depends on the design of the interventions and the effects they will have on the structure. If the interventions are likely to modify the global response of the structure, a global analysis is required: each macroelement must be evaluated in terms of ground acceleration at the ultimate limit state, both before and after the interventions. In fact, at the initial state the horizontal seismic action is distributed among the various macroelements, but the structural interventions may modify the original seismic distribution and lead to negative effects on some other macroelements. From the other hand, if the intervention involves a limited area of the structure, the seismic evaluation may be confined to the analysis of the relevant macroelements, being the global evaluation of the church redundant and problematic.

5.3.1 Choice of the local mechanism

A global seismic analysis of the church of St. Constantine & Helen was performed as described in [15]. The structure was modeled by eight-node solid elements with three degrees-of-freedom per node, four-node shell elements were used to model the central dome, while frame elements were used for the proper connection and collaboration of solid and shell elements. The analysis, a modal response spectrum analysis, was conducted for the structure at the initial state and after the strengthening interventions.

In this chapter, a local seismic analysis to assess the transversal response of the church will be presented. As shown in Figure 2 the collapse mechanism for this macroelement may occur for cracking in the arches, rotation of the walls, out of plumb or crushing in the columns. The structural evaluation will be carried out referring to the state before and after the intervention of grout injection in masonry.

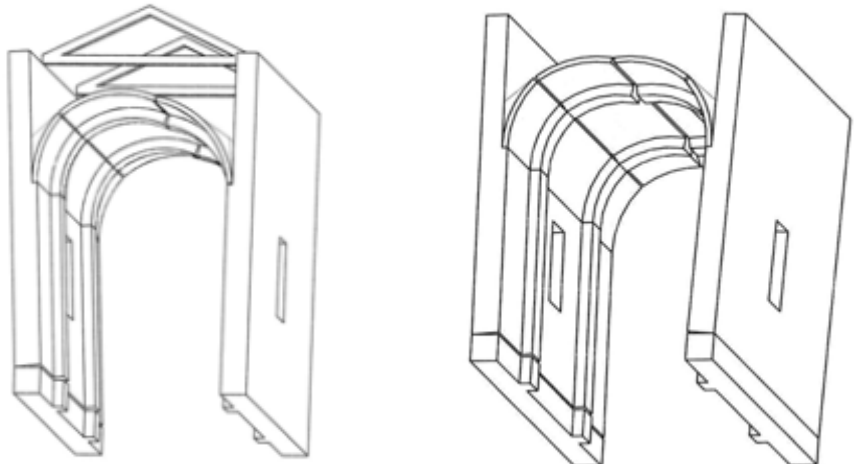


Figure 2. Collapse mechanism with respect to the transversal response of the church

5.4 STRUCTURAL MODELING

The macroelement analyzed consists of three arches and the four piers arranged along an alignment in the transverse direction of the church. The structure was modeled by frame elements following two methods provided by the software Aedes PC.E: the "Conci e giunti" method to model the arches and another method for the modeling of the masonry pillars.

5.4.1 "Conci e giunti" method

The system is particularly appropriate for the modeling of arches within complex structures.

According to limit analysis approach, the arch can be assessed in a conservative way as a rigid-brittle system: the arch is fixed at the base and modeled as a set of rigid elements, the connection between them is provided by three truss elements, two of them reacting to axial force and one reacting to shear. However, in case of complex systems that include the structures adjacent to the arch, this method can hardly be adopted. Considering a rigid-brittle behavior of the arch the elasticity of the system is not taken into account, therefore modal analyses cannot be performed and the non-linear behavior cannot be assessed in terms of variation of the internal constraints.

This issues can be overcome with the "*conci e giunti"* method, a finite element method implemented by the software Aedes PC.E, that models the arches as a set, respectively, of blocks and joints. As shown in Figure 3, both blocks and joints are frame elements: the blocks have the mechanical properties of the stone and their cross section is the effective cross section of the arch, the joints represent the mortar between the blocks, there are four joints for each block therefore the joint cross section is ¼ of the arch cross section. The joints have a pin end and fixed end, the fixed end provides continuity with the previous block while the pin end transfers shear and axial force to the next block. The joints are connected to the block ends through rigid links. The all system transfer moment, shear and axial force from one block to another.

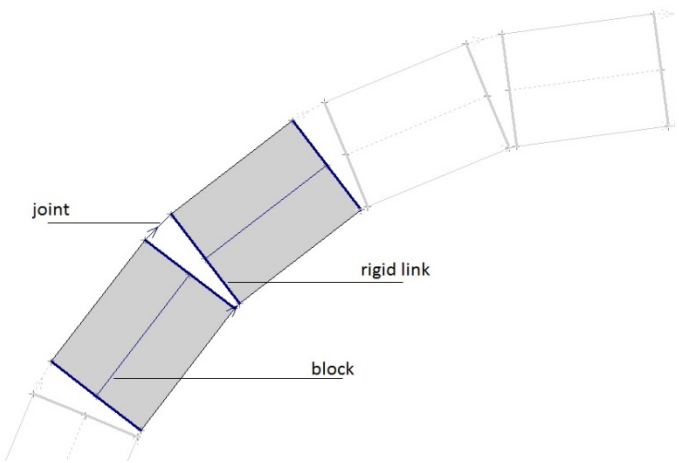


Figure 3. Structural model of an arch

Performing a pushover analysis, the joints are subjected to verification in terms of axial force: if the tensile stresses exceed the resistance, the fixed end is turned to pin and the axial force is released so that the element loses any stiffness, the internal actions remain constant at the value reached so far. As the analysis continues the progressive deterioration

of the joints will determine an unstable configuration that define the end of the pushover curve.

The tensile strength of the material can assume a finite value or be null. In case of zero tensile strength a comparison with the results of a limit analysis is feasible. However the possibility to define a finite value of the tensile strength of the material prevents from underestimating too much the capacity of the structure.

5.4.2 Masonry pillars

The pillars are divided into blocks and modeled as frame element. At each step of the pushover analysis the blocks are subjected to verification in terms of compression checking the position of the pressure curve. If the pressure curve comes out of the section of the block, a plastic hinge develops in the relevant node. The incremental analysis continues until the structure becomes unstable determining the maximum value of the lateral force.

5.4.3 Collapse multiplier

The aim of non-linear static analysis applied to the *"conci e giunti"* system and to the masonry pillars is to determine the collapse multiplier. The analysis plays the same role of the kinematic analysis but the collapse multiplier is pursued as the maximum static multiplier instead of the minimum kinematic multiplier. The collapse multiplier is calculated with the following formula:

$$\alpha_0 = \frac{F_{max}}{W} \tag{1}$$

where:

F_{max} is the value of the maximum lateral force the structure can sustain;

W is the total weight of the structure.

The spectral acceleration that activates the mechanism can be calculated as follow:

$$\alpha_0^* = \frac{\alpha_0 \cdot g}{e^* \cdot CF} \tag{2}$$

where:

g is the gravity acceleration;

e^* is the participating mass ratio;

CF is the confidence factor related to a knowledge level KL1, therefore equal to 1.35.

The participating mass and the participating mass ratio can be evaluated with the following formulas considering the virtual displacement of the nodes where the weight is applied:

$$M^* = \frac{\left(\sum P_i \delta_{x,i}\right)^2}{g \cdot \sum P_i \delta_{x,i}^2} \tag{3}$$

$$e^* = \frac{g \cdot M^*}{\sum P_i} \tag{4}$$

where:

P_i is the weight applied to node i of the structure;

$\delta_{x,i}$ is the virtual horizontal displacement of node i at the ULS.

5.5 ANALYSIS RESULTS

The software Aedes PC.E was used to create the model and perform the analyses. The model shown in Figure 4 consists of 642 frame elements and 526 nodes. The structured is analyzed in its plane, for each node the rotation around the vertical z axis is restrained while the nodes at the base of the piers are fully fixed.

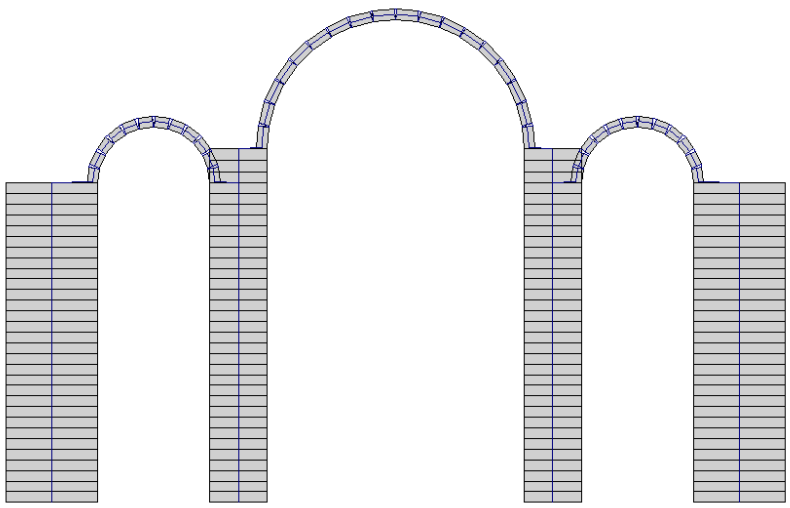


Figure 4. Structural model of the macroelement analyzed

To account for the weight of the structure that was not included in the model, vertical point loads have been applied at the top nodes of the piers. Their value were calculated in order to have a vertical reaction at the base of the pier equal to the one found with the global model of the church: 630 kN were applied on the lateral piers while 605 kN were applied on the central ones. The total weight of the structure is:

$$W = 4824\ kN$$

5.5.1 Modal analysis

A modal analysis was performed for the structure at the initial state and after the intervention of grout injection. Figure 5 shows the deformed shape of the first mode of vibration, the red spheres represent the mass associated to each node.

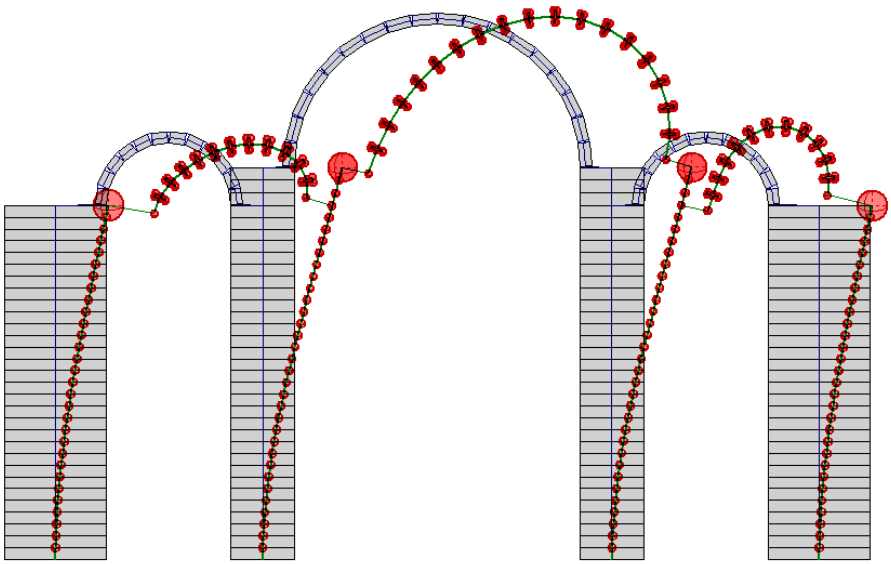


Figure 5. Deformed shape of the fundamental mode of vibration

Table 1 shows the fundamental period of the structure in the two situations analyzed and the participating mass associated to that mode.

Table 3. Fundamental period of the structure

	Period (s)	Participating mass (%)
At initial state	0.519	76.3
After interventions	0.443	74.1

5.5.2 Pushover Analysis

The pushover analyses were performed using two distribution of the lateral loads:

- a “triangular” distribution, based on lateral forces proportional to mass and elevation;
- a “uniform” distribution, based on lateral forces proportional to mass regardless of elevation.

Figure 6 shows the capacity curves of the structure at the initial state, obtained for the load distribution A and B. In the first case the structure reaches an unstable configuration for a value of the lateral force equal to 310 kN, while with load distribution B the maximum lateral force the structure can sustain is 600 kN.

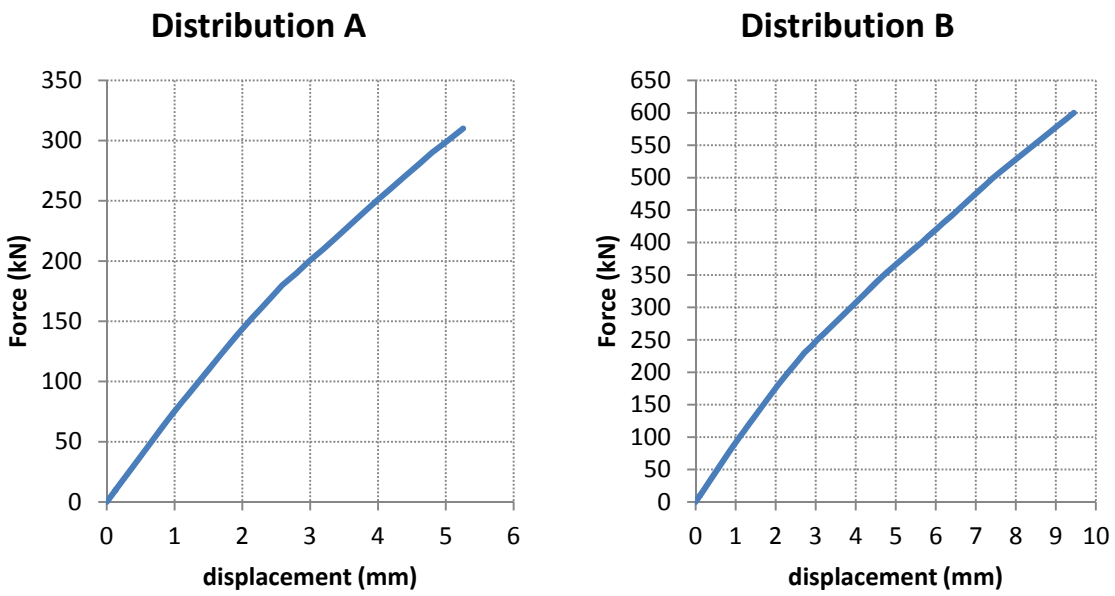


Figure 6. Capacity curves of the structure at initial state

The results obtained with the "triangular" distribution of the lateral forces are more severe and will be considered as representative of the behavior of the structure for the safety verification. Figure 7 shows the deformed shape and the position of the pressure curve at the last step of the pushover analysis with distribution A.

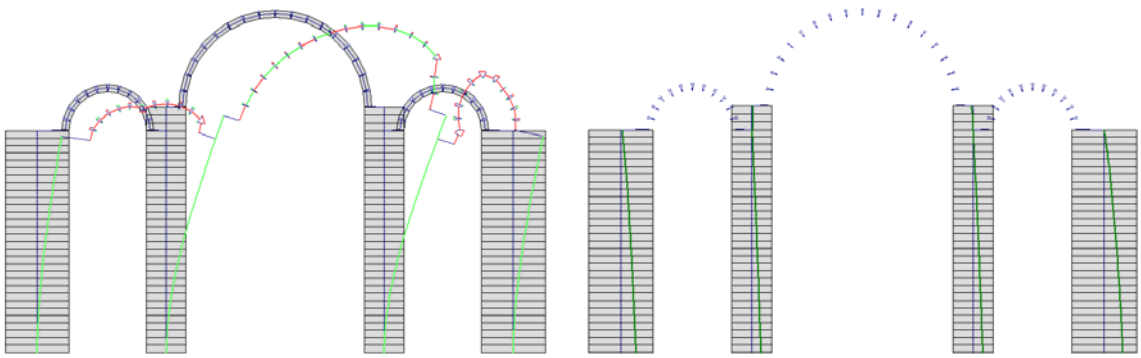


Figure 7. Deformed shape and pressure curve at last step of curve A

The deformed shape reveals the position of the plastic hinges that developed in the arches for the degradation of the joints between the blocks. The pressure curve remain internal to the cross section of the piers, therefore no plastic hinges develop in that area.

Finally, the value of the collapse multiplier is:

$$\alpha_0 = \frac{F_{max}}{W} = \frac{310}{4824} = 0.064$$

The participating mass ratio was calculated according to Formula 4 and its value is e*=0.686. Therefore, the acceleration that activates the mechanism, calculated with formula 2, is:

$$\alpha_0^* = \frac{\alpha_0 \cdot g}{e^* \cdot CF} = \frac{0.643}{0.686 \cdot 1.35} \cdot g = 0.069\, g$$

The same pushover analysis was performed after the intervention of grout injection. The material properties were adjusted according to the values presented in 5.2. Figure 8 shows the capacity curves of the structure for the two load distribution of the lateral forces. In the analysis with the triangular distribution the structure sustained a maximum lateral force of 600kN, while with the uniform distribution the structure reaches an unstable configuration for a lateral force equal to 700 kN.

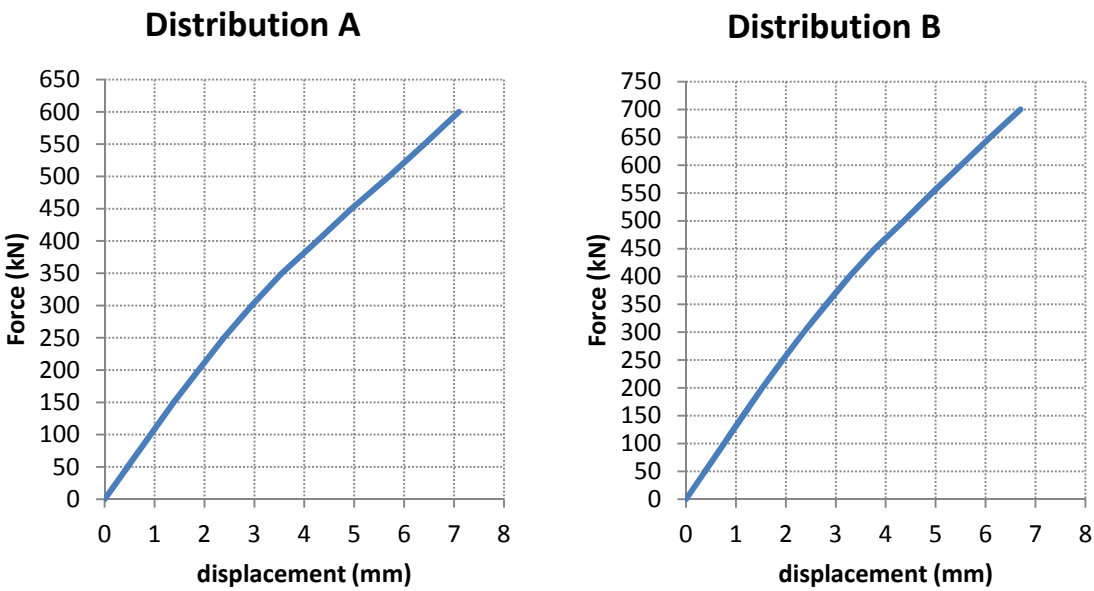


Figure 8. Capacity curves of the structure after interventions

Even in this case the maximum lateral force obtained with load distribution A is smaller than the one obtained with load distribution B and it will be used for the safety verification. Figure 9 shows the deformed shape and the position of the pressure curve at the last step of the pushover analysis.

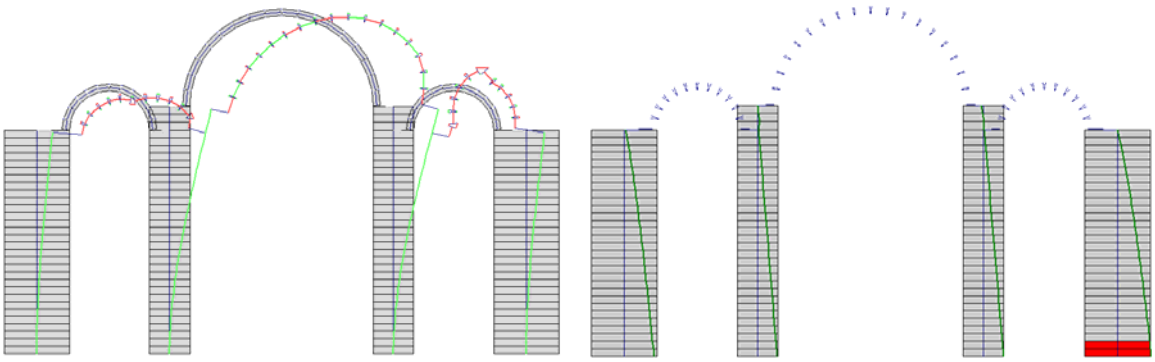


Figure 9. Deformed shape and pressure curve after the interventions

In addition to the plastic hinges that developed in the arches, the position of the pressure curve comes out from the cross section of the right pier. Therefore plastic hinges developed at the relevant nodes at the base of this pier.

After interventions, the value of the collapse multiplier is:

$$\alpha_0 = \frac{F_{max}}{W} = \frac{600}{4824} = 0.124$$

The participating mass ratio was calculated according to formula 4 and its value is e^*=0.657. Therefore, the acceleration that activates the mechanism, calculated with formula 2, is:

$$\alpha_0^* = \frac{\alpha_0 \cdot g}{e^* \cdot CF} = \frac{0.124}{0.657 \cdot 1.35} \cdot g = 0.140\ g$$

5.6 SAFETY VERIFICATION

According to the Italian Structural Standards (C8A.4.2.3) [4] the verification with respect to the ultimate limit state is satisfied if the spectral acceleration α_0* that activates the mechanism satisfies the following:

$$\alpha_0^* \geq \frac{\alpha_g \cdot S}{q} = \frac{0.16g \cdot 1.2}{1.5} = 0.128\ g$$

where:

α_g is the design ground acceleration for ground type A, equal to 0.16g ;

S is the soil factor, equal to 1.2;

q is the behavior factor, assumed equal to 1.5.

For the structure at the initial state:

$$\alpha_{0,1}^{*} = 0.069g < \frac{\alpha_g \cdot S}{q}$$

The structure doesn't satisfy the check and the maximum ground acceleration it can sustain is:

$$\alpha_{g,max,1} = \alpha_{0,1}^{*} \cdot q/S = 0.086g$$

After the intervention of grout injections:

$$\alpha_{0,2}^{*} = 0.140g > \frac{\alpha_g \cdot S}{q}$$

Therefore, the structure satisfies the check and the maximum ground acceleration it can sustain is:

$$\alpha_{g,max,2} = \alpha_{0,2}^{*} \cdot q/S = 0.175g$$

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