Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2017) 2619-2638

ECCOMAS
Proceedia

COMPDYN 2017
6th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15–17 June 2017

SEISMIC MODELLING AND ANALYSIS OF MASONRY BUILDING IN AGGREGATE: A CASE STUDY

Siro Casolo[1], Carlo Alberto Sanjust[1], Giuseppina Uva[2] and Vito Diana[1]

[1] Dep. ABC, Politecnico di Milano

P.zza L. da Vinci 32, 20133 Milano, Italy

e-mail: {siro.casolo, carlo.sanjust, vito.diana}@polimi.it

[2] Dep. Dicatech, Politecnico di Bari

Via E. Orabona 4, 70125 Bari, Italy

giuseppina.uva@poliba.it

Keywords: Masonry buildings, Building Aggregates, Non-linear Dynamics.

Abstract. *The reuse of historical centers requires to solve in a reliable manner the problem of seismic analysis of the residential units belonging to building clusters. This implies complex interactions at the structural level, which influence the seismic response and damage mechanisms of the individual units. Even if a rigorous approach would require the analysis of the entire aggregate, in the professional practice, the safety assessment is typically performed on the single unit [1]. At the regional scale, empirical methodologies aimed at taking into account the variation of the vulnerability according to the position of the unit within the aggregate and some geometrical features were developed. In the case studies here presented, a computational modelling in the non-linear dynamic field is performed, which allows to calibrate the fragility curves for the building aggregate. The case studies are two large building clusters located in Puglia (in the historical centers of Foggia and Sant'Agata di Puglia [2]). Preliminary FEM simulations under seismic actions provided information about the overall response in terms of vibration and deformation modes, allowing to select the most significant ones. Then, it was possible to define a simplified plane model, able to represent the transversal response, which revealed to be the predominant one. Moreover, some partial, simplified models were defined for limited architectural parts that are particularly representative regarding the development of mechanisms of damage and collapse [2]. These models, consisting of an assembly of quadrilateral elements, are formulated in accordance with a RBSM approach [3,4]. Non-linear dynamic analyses have shown that the vulnerability is very different from the one predicted by classical vulnerability methods, highlighting that in the case of complex cluster, a more detailed and specific tool for the vulnerability analysis is required.*

Peer-review under responsibility of the organizing committee of COMPDYN 2017.
doi: 10.7712/120117.5593.18376

1 INTRODUCTION

The paper presents a research study concerning the regional seismic vulnerability assessment of the building stock in the historical centres of the Province of Foggia (Puglia, Southern Italy). It was part of the Research Project "*Antaeus*" funded by Regione Puglia (Fund CIPE 20/2004) and managed by "*Autorità di Bacino della Puglia*" (Basin Authority of Puglia) in cooperation with a number of Public Institutions (Department Dicatech of the University "Politecnico di Bari", Municipality of Foggia, Administration of the Province of Foggia). The general objective of the project was to provide the local administration with multi-level methodologies and tools for the seismic vulnerability assessment of the regional territory [2]. After developing and testing vulnerability methods at the regional scale, a detailed study was performed on masonry aggregate buildings in the City of Foggia. In particular, the paper presents the results of a series of mechanical analysis performed first by Finite Element Modeling [5] and then by a specific Rigid Body and Spring Model [3,4,6,7]. As a case study, a representative structural typology of the ancient historical center of Foggia was chosen, with the objective of characterizing the seismic response of local building types and allowing the validation of the 1st level vulnerability assessment already carried out in the Project but also, in the future, the derivation of specific fragility curves.

1.1 Seismic vulnerability assessment at the regional scale: reference framework and peculiarity of the local application context (Puglia)

The assessment of the seismic vulnerability of a territory is one of the crucial elements of seismic risk prevention and mitigation strategies which are the challenges that are being faced in the last decades. Many recent earthquakes have clearly shown that the existing building stock in many countries all over the world is severely at Risk, and that in many cases the politics adopted in the past have been inadequate or completely absent, leaving space to extemporaneous actions, with ineffective or even detrimental effects. These questions are – first of all – a political and administrative matter, but also directly involve the scientific community, that is required to develop the theoretical reference framework and effective operative tools. A structural vulnerability analysis involves the assessment of the consistency of the existing building stock in a given area both from a qualitative and quantitative point of view, with a specific regard to the propension of buildings to suffer damage after an earthquake. First of all, a vulnerability methodology should specify how the inventory of the buildings has to be carried out, establish the desired level of detail and develop suitable models for the correlation of the ground motion severity and the possible physical damage and losses, both tangible and intangible. Finally, the buildings shall be classified per a priority list. It is important to point out that the application of any kind of method is conditioned by the great amount of basic data and the relevant computational effort is required for the assessment at the scale of the building. If the methodology is to be applied to a large urban area, it is then appropriate to single out a set of typological models able to represent a plurality of buildings, and to adopt some simplifications in the approach. In the past twenty years, two different approaches for the seismic vulnerability assessment at the geographic scale have been developed, generally known as 1st Level and 2nd Level approach:

- 1st level procedures are aimed at a preliminary evaluation based on few empirical parameters, and input data are limited to information that can be gathered by simple and quick visual inspections [8-13];
- 2nd level procedures include more detailed elements about the structural characteristics and damage modes. They always do operate at a territorial or urban scale, but

are usually specifically devoted to a building type (churches, palaces, bridges,. . .) and collect much more detailed data [14-18].

It is evident anyway that the results provided at a large scale (both regional and urban) have only a relative validity within the considered set of buildings (which shall be sufficiently “homogeneous” regarding the typological, structural and constructive aspects). This guarantees that the previsions of the models, although approximate, will have a relative validity within the considered set. It will be possible to sort by vulnerability/risk level the elements of the reference set in order to budget the different intervention options and support the definition of mid and long term mitigation strategies, whereas comparing the results obtained on very different geographic areas could be misleading. Finally, the actual safety level of an individual building can be obtained only by means of a complete structural calculation of the building, that is sometimes referred to as the 3rd level analysis [19-22].

2 THE CASE STUDY

The object of the vulnerability study is the historical center of the City of Foggia (Puglia, Southern Italy – Fig.1). Most part of the old town is the result of the reconstruction work that took place after the earthquake of 1731, within the two central districts of “*Borgo Croci*” and “*Quartieri Settecenteschi*” (Fig. 1).

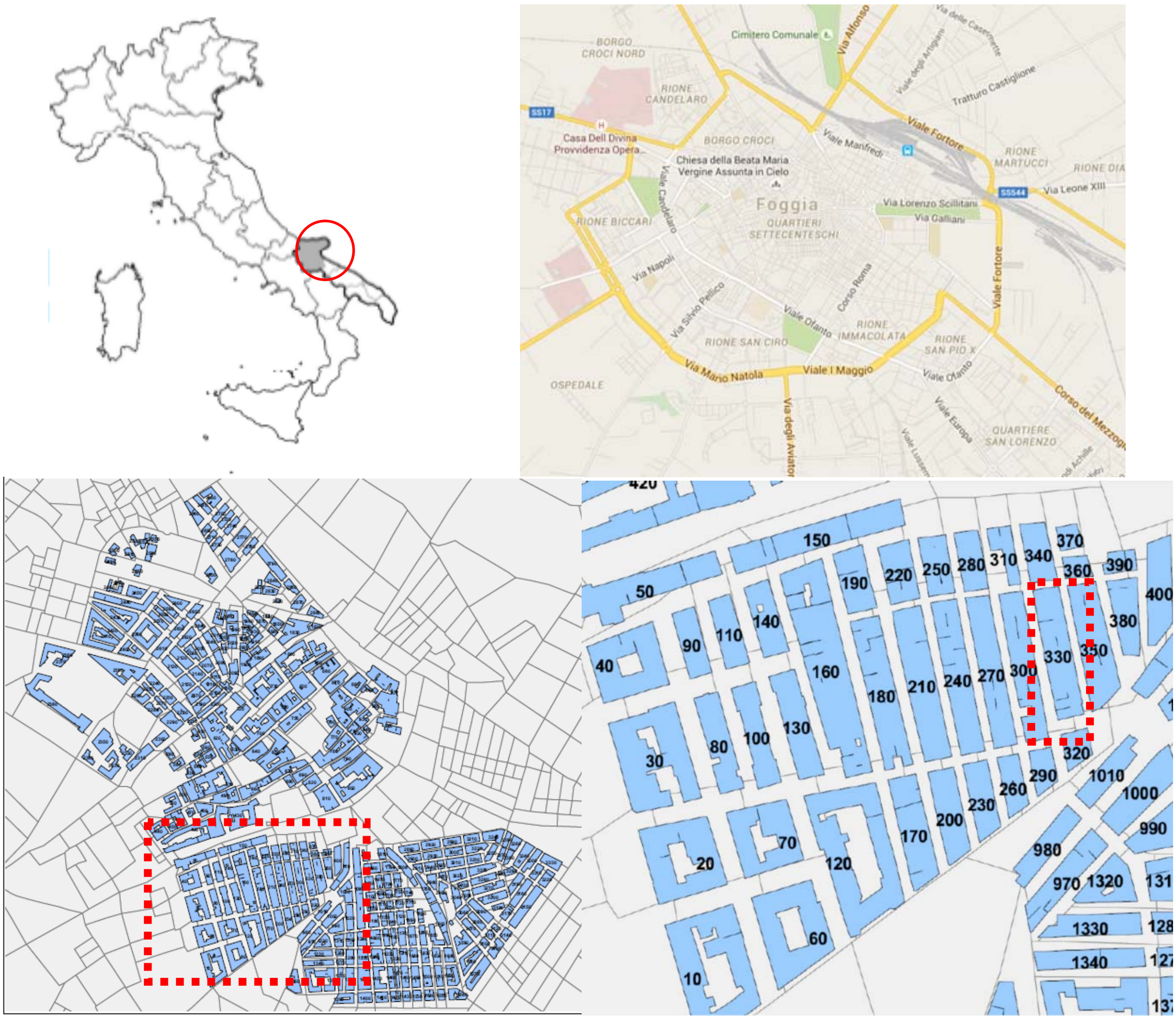


Figure 1: Map of the historical center of Foggia; position of the selected building aggregate (#330).

The building aggregate selected for the analyses (its position on the map of the old town of Foggia is shown in Fig. 1, where it is marked with the number *330*) well represents the most widespread structural type in the historic center of the city of Foggia, and is moreover highly vulnerable. In these buildings, the first floor dates back to the reconstruction that took place after the devastating earthquake of 1731.

Figure 2: Plan view and prospect views from the main streets.

The geometry of the building aggregate – an elongated rectangle - is quite regular in plan, as it can be observed from the plan of the ground floor shown in Fig. 2-top.

Figure 3: General views of the building aggregate from the two sides.

On the other side, a situation of severe irregularity in elevation is observed (Fig. 2, 3), mainly due to two factors:

1) there is a widespread and chaotic presence of superelevations, new floors and roofs added to the original layout of the ground floor, which dates back to the reconstruction that took place after the devastating earthquake of 1731.
2) the position of openings on the main façades facing the main streets is very irregular and inconsistent, especially in correspondence of later superelevated floors.

Figure 4 shows the results of the 1st level vulnerability investigation performed during *Antaeus Project*. From these figures, it can be observed that most of the structural units surveyed near the building aggregate chosen as case study present a vulnerability index comprised within the range *0.4-0.6* (the range of the index is [0-1]), represented in yellow. More specifically, the vulnerability index of the structural units belonging to aggregate 330 is almost completely homogeneous, with some exceptions. In particular, it is worth noting that there is an increase of vulnerability in the corner structural units, whereas some units located in the middle, and with no superelevated floors exhibits lower vulnerability indexes.

Figure 4: View of the vulnerability map of the City of Foggia in the District "*Quartieri Settecenteschi*" and detail on the building aggregate #330.

Of course, the quick vulnerability assessment through survey forms does not allow to obtain an indication about the safety level of the individual building, but rather should be considered as a relative value within an homogeneous set of building units. It should also be considered that in these kind of investigations, the reliability of results strongly depends on the human factor: data are collected via direct surveys performed by trained teams of technicians, and being based on subjective judgement, are affected by possible misunderstanding. With regard to this aspect, it is interesting to analyze the map of Fig. 5, which shows the reliability of the survey teams, based on the completeness and quality of data reported in the forms.

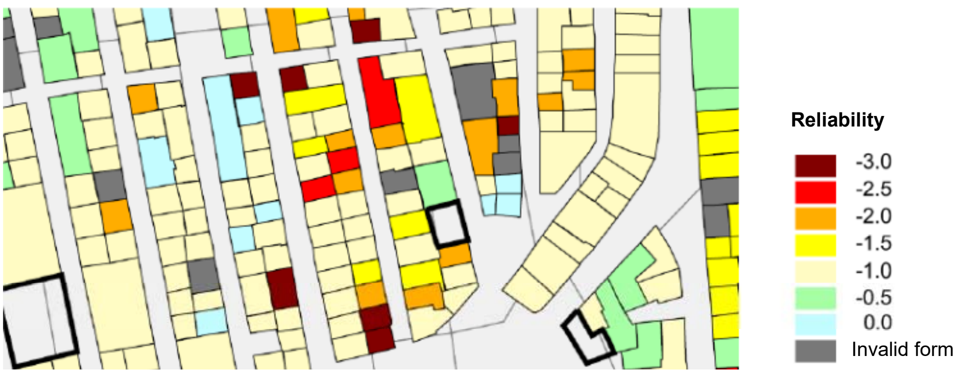


Figure 4: Reliability map of the City of Foggia in the District "*Quartieri Settecenteschi*" and detail on the building aggregate #330.

In the Figure, the scale ranges from 0 to -3.5 (the score 0 corresponds to maximum reliability, and negative scores progressively indicate lower reliability). It can be observed that several structural units within the building aggregate #330 present a very low reliability index, that is to say, the limitations of vulnerability index estimates obtained through rapid surveys should be always kept in mind, and when the objective is to derive specific indications about the vulnerability of individual units, more detailed investigations and models must be introduced.

3 COMPUTATIONAL MODELING OF THE SEISMIC BEHAVIOUR OF THE BUILDING AGGREGATE: FE MODEL

The numerical modelling is based on detailed data and information provided by the technical office of the Municipality of Foggia. A full survey, including detailed plan views of the different floors, section views, photographic documentation.

3.1 Geometry and characteristics of the materials

In order to understand the overall behavior a numerical model describing the geometry of the entire building aggregate has been implemented, by adopting the finite element code "*Abaqus*" [5]. The objective of this first FE model is to obtain useful information to characterize both the static behavior under the effect gravity loads and the dynamic behavior, in terms of natural vibration modes.

The geometry has been simplified as much as possible, both to contain the number of variables of the numerical problem and to facilitate the subsequent processing and interpretation and of the results obtained. The objective, in fact, was not to realize a faithful reproduction of the specific aggregate, but rather to grasp its essential and characteristic features, the essential characteristic, which can be considered representative of a typical and recurrent cases within the historical center of Foggia

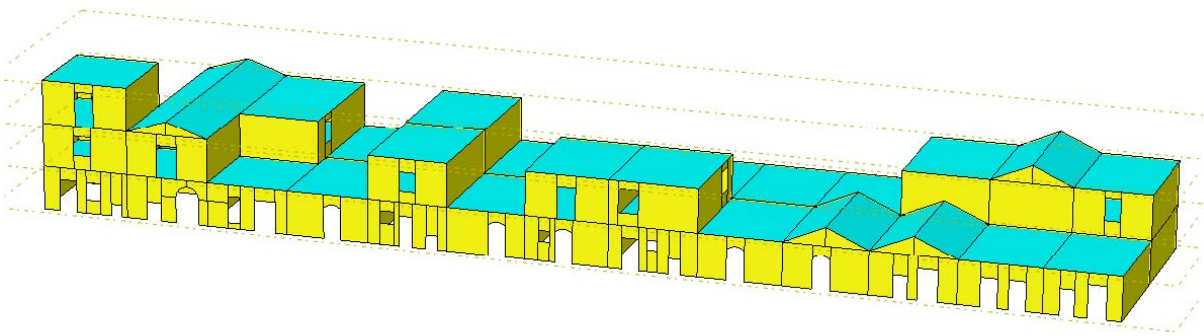


Figure 5: Overall geometry of the FE model. Vertical structural elements are shown in yellow, whereas horizontal structures and roofs are shown in blue.

Figure 5 shows the axonometric view of the simplified geometrical FE model of the aggregate building. In order to evaluate the overall behavior in the linear elastic field, two materials have been defined, one for vertical structures (tuff masonry, represented in yellow in Fig.5), and one for horizontal structures and roofs (represented in blue in Fig.5). The first material employed (vertical structures) is orthotropic, with the following material properties: mass density $\rho=1800\ kg/m^3$, Young's Modulus $E=900\ MPa$ (for all directions); Poisson's ratio $\nu=0.05$; Shear Modulus $G=150\ MPa$ (for all directions). The orthotropy of the material has been exploited in order to define a shear deformability much lower than the one theoretically allowed in the case of isotropic material. With regard to the second material (horizontal structures and roofs), it is assumed to be isotropic, with the following material properties: Young's Modulus *E=3000 MPa*; Poisson's ratio ν=0.05; and mass per unit area *300 kg/m²*.

3.2 Discretization of the geometry

The discretization of the overall geometry was made by using plane finite elements of "shell" type, which in Abaqus are labelled with the code *S4R*. Figure 6 shows two views of the model, in which the mesh according 4-nodes S4R elements can be seen.

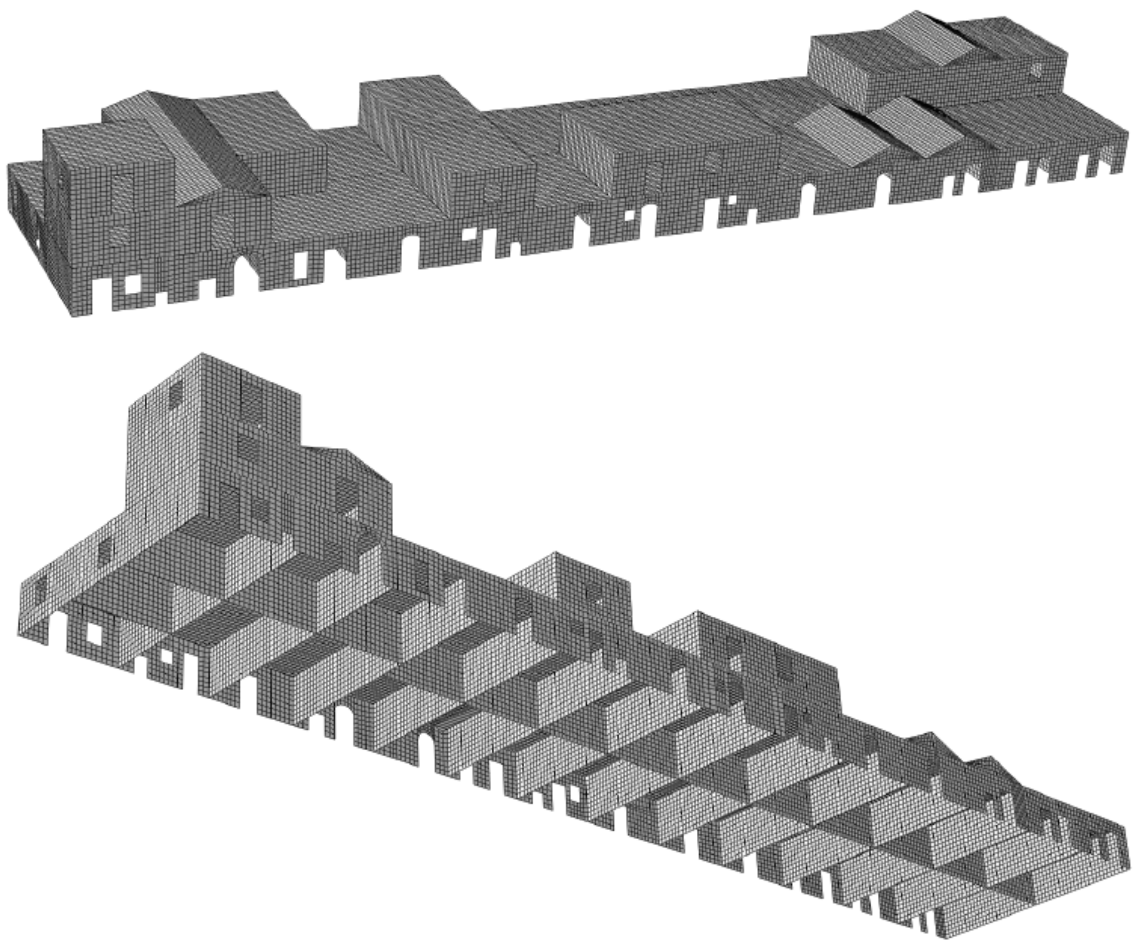


Figure 6: Axonometric views of the FE model.

The FE model of the entire aggregate comprises a total number of 44783 nodes and 44182 elements, for a total of 268.698 degrees of freedom. At the base, elements have a fixed constraint to the ground. The possible presence of excavated underground levels has been neglected.

3.3 Analysis of the 3D Model under gravity loads

The analysis under gravity loads provides stress values which are quite low, as it could be expected considering the low height. Figure 7 shows the map of the vertical component S_{22}, where it can be observed that only at the base the values reach *0.1 MPa*.

It should be noted that this preliminary analysis does not indicate any particularly critical situation, even where the distribution of openings is particularly irregular in elevation.

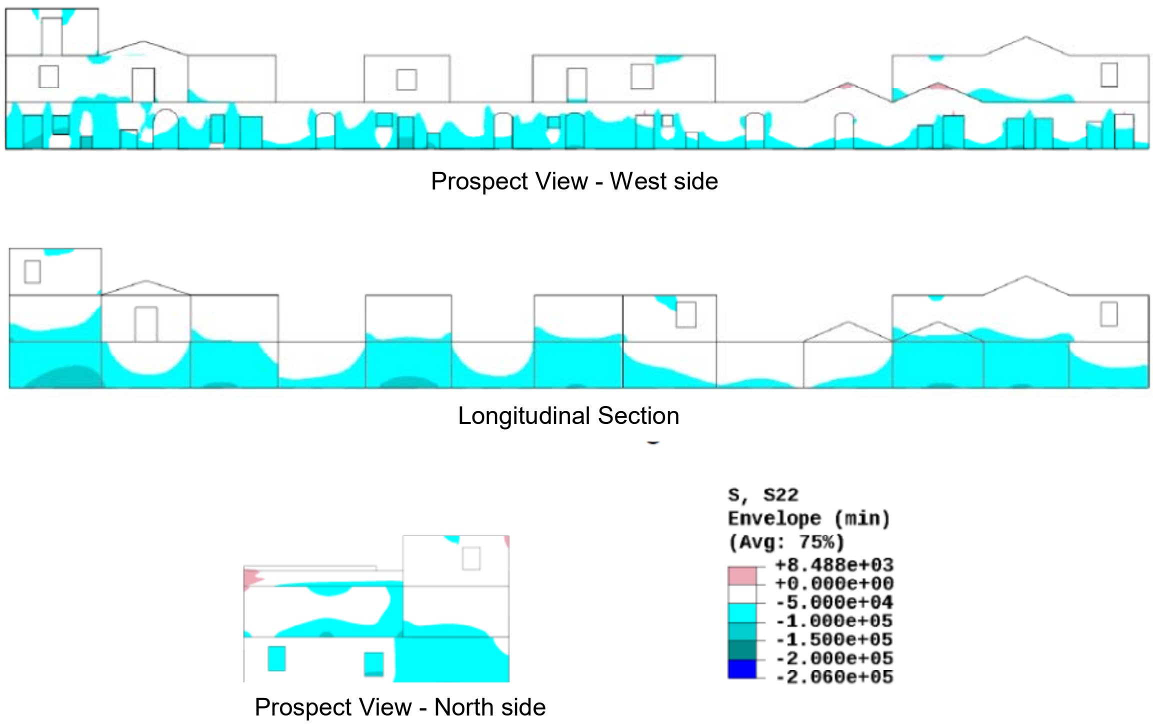


Figure 7: Map of the S22 stress component under gravity loads [N/m^2].

3.4 Modal Analysis: Eigenvalues and eigenvectors

The modal analysis of this model shows that it is not easy to clearly identify a vibration mode that involves the entire building aggregate, because of the complexity of the geometry and the presence of many irregularities in elevation. Figures 8 and 9 show the first five modal shapes with the indication of the value of the corresponding period T_i.

P A R T I C I P A T I O N   F A C T O R S

MODE NO	X-COMPONENT	Y-COMPONENT
1	2.1009	-0.82675
2	1.5725	1.5886
3	1.9272	-0.37588
4	-0.25028	0.80313
5	0.25344	2.1156
6	0.53319	0.49592
7	-8.77282E-02	0.68968
8	0.25693	8.37329E-02
9	0.30111	-0.12236
10	-0.85928	-0.40738

E F F E C T I V E   M A S S

MODE NO	X-COMPONENT	Y-COMPONENT
1	392465.	60778.
2	256942.	262211.
3	791413.	30106.
4	24334.	250573.
5	12072.	841261.
6	35859.	31020.
7	1463.	90401.
8	5817.	618.
9	19453.	3212.
10	71533.	16078.

Table 1: Modal participation factors and effective masses in the two horizontal directions.

Table 1 summarizes the Modal Participation Factors and the Effective masses in the two horizontal directions, as calculated by the code Abaqus [5]. From these data, it can be noted that for modes higher than the 5th, Modal Participation Factors and Effective Masses are very low, and therefore such modes are not significantly excited by the ground motion. By looking at the deformed modal shapes (Figs. 8, 9), it can be concluded that the mode that significantly involves the entire aggregate is the 5th, differently from regular structures, for which this happens for the shortest period mode. Mode 5 is characterized by oscillations directed along the transversal and short direction (Y), and has the largest value of effective mass. On the other side, the modal analysis points out that the motion becomes very irregular and complex for the end structural units, especially at the short northern front. In all the modal shapes, the most critical parts of the entire structure tend to be always those which correspond to the two short ends of the aggregate.

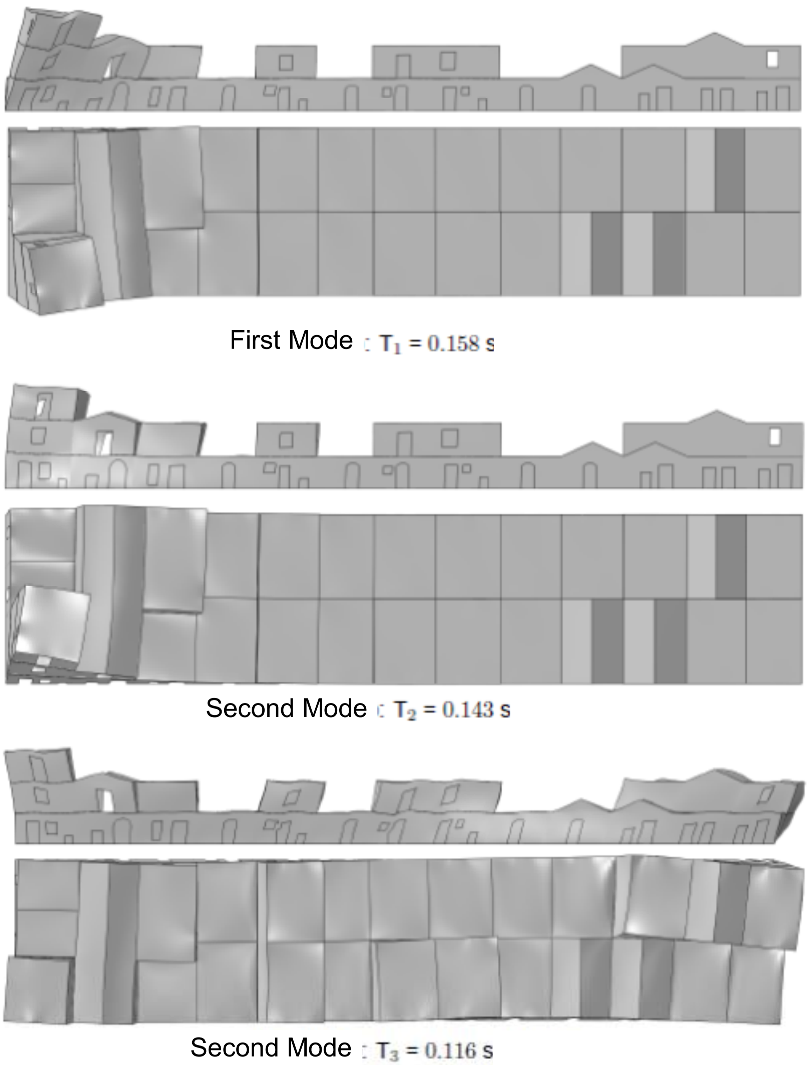
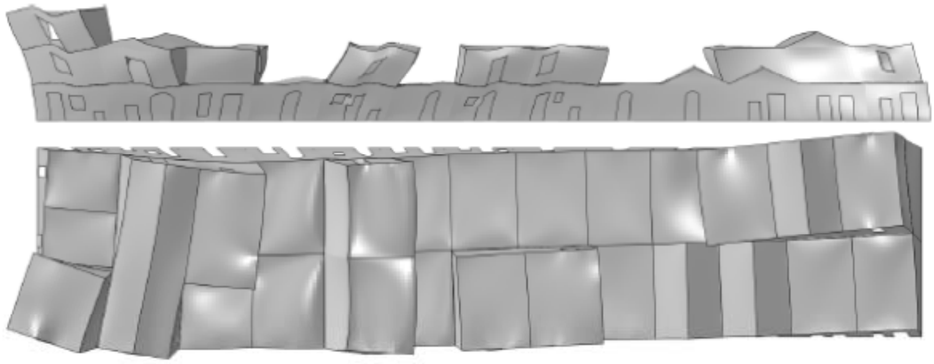
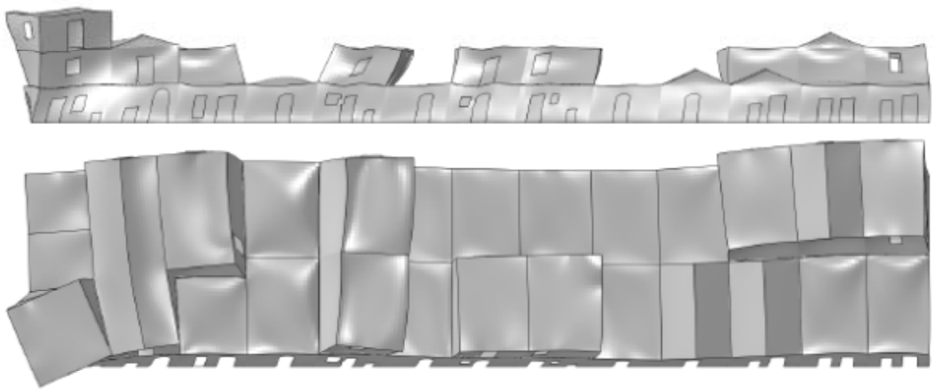


Figure 8: Eigenvalues analysis: deformed shapes and periods corresponding to the first three modes.

.



Fourth Mode : $T_4 = 0.108$ s



Fifth Mode : $T_5 = 0.105$ s

Figure 9: Eigenvalues analysis: deformed shapes and periods corresponding to the 4th and 5th modes.

Those located at the north side appear to be the most stressed, both with respect to motions having the prevalent component oriented along the longitudinal of the aggregate (X component), and along the transverse direction (Y component). The presence of a quote of torsional movement can also be observed, which is the consequence of the asymmetry and irregularity in elevation.

4 THE RBS MODEL

4.1 The modeling approach

In order to perform a quantitative assessment of the seismic response of the case study under potentially destructive seismic events, it was necessary to perform nonlinear dynamic analyses, considering the non-linear behavior and damage development of structural elements.

The complete three-dimensional FE model described in the previous section was unsuitable to this scope, both for the difficulty of implementing a proper constitutive model of masonry, both because of the high number of degrees of freedom, equal to 268698, which makes nearly impossible the performance of dynamic analyses from the point of view of the required computational resources.

These analyses have been performed by means of a specific mechanistic model, made by rigid masses and springs, (RBSM) which considers only the in-plane dynamics. This model can describe higher vibration modes, as well as the combined axial and shear deformation and damage of the material by means of a simplified heuristic approach, which has been developed both for the out-of-plane behaviour [23-27] and for the in-plane behaviour [3-4, 6-7] here applied. The application of the mentioned RBSM model is particularly effective since it allows to considerably reduce the computational burden while keeping, at the same time, the information about the meso-scale effects of the masonry behavior. It has been, in fact, fruitfully employed both for the analyses of complex, real buildings [19-22] and when it is used to perform a large number of analyses for calibrating simplified macro-models of masonry panels [28,29].

4.2 Definition of representative plane models

In order to perform a quantitative assessment of the seismic response of the case study under potentially destructive seismic events, it was necessary to perform non linear dynamic analyses, considering the non-linear behavior and damage development of structural elements.

The complete three-dimensional FE model described in the previous section was unsuitable to this scope, both for the difficulty of implementing a proper constitutive model of masonry, both because of the high number of degrees of freedom, equal to 268698, which makes nearly impossible the performance of dynamic analyses from the point of view of the required computational resources.

Thence, proper 2D "sub-models" have been identified, in which the geometry is plane and simplified, but the mechanical model can describe the development of the relevant mechanisms of damage and collapse [30]. Actually, this approach is often the only real possibility, provided that the simplified model is properly calibrated so as to make it dynamically equivalent and able to describe the kinematics of the case study. Some applications of this approach to the computational modelling of masonry historical buildings characterized by a great geometrical complexity can be found in the scientific literature [31].

Three plane models capable to represent the kinematic highlighted in modal analysis have been defined. In particular, the first two models are intended to represent the response of the North end-unit along X and Y directions.

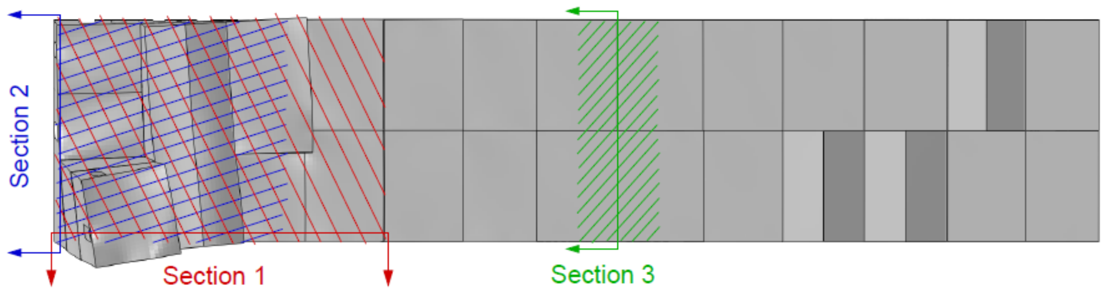


Figure 10: The sections used to extract the plan models, with indication of the 3D pertaining area.

In Figure 10 the two sections, called "Section 1" and "Section 2", are represented, showing in blue and red, respectively, the area pertaining to the three-dimensional model. The motion that involves the entire aggregate in the transverse direction is described by means of a model extracted from the "Section 3" (Fig. 10). From the above-mentioned Sections, three corresponding computational plane models have been defined, by an assembly of quadrilateral elements having the geometry shown in Figure 11. These models are formulated in accordance with the aforementioned RBSM approach.

It is worth remembering that the objective is not to reproduce the actual behavior of the specific case study, but to evaluate by in quantitative terms the seismic vulnerability of a representative structural type of the city of Foggia.

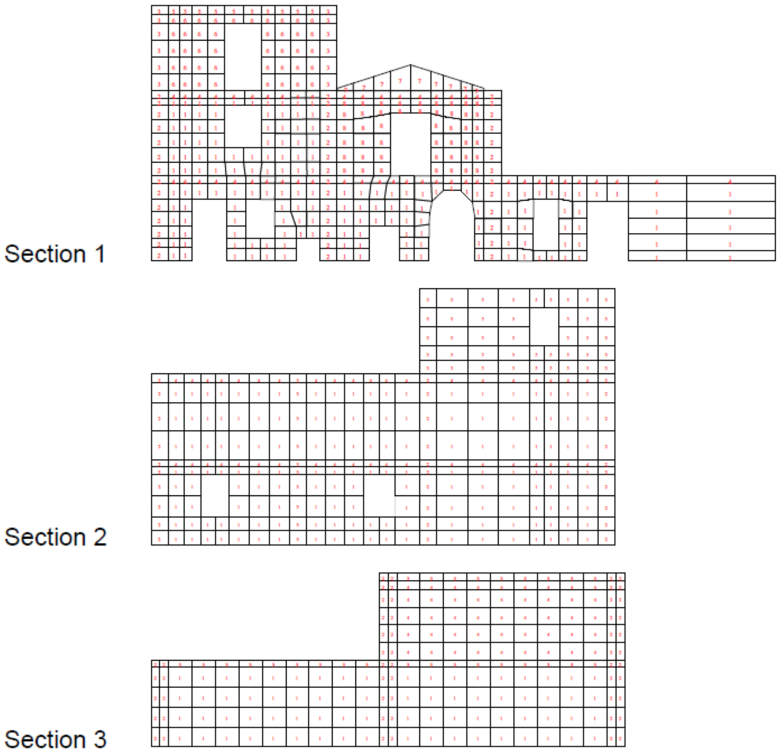


Figure 11: The RBS plane models corresponding to the sections shown in Figure 10.

A summary of the characteristics of the three models in terms of engagement of computing resources, and therefore of practical feasibility of many dynamic analyses in the non-linear field, is reported in Table 2.

	Section 1	Section 2	Section 3
Number of elements	377	273	198
DoF	1131	819	594
Number of springs	2031	1557	1155
Band width	62	47	35
Total mass	733	876	181

Table 2: Characteristics of the three RBSM numerical models.

In Figure 12, the scheme of the constitutive behaviour used for the materials is shown.

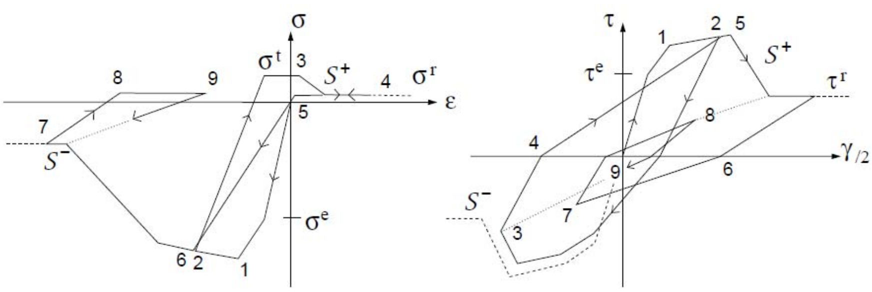


Figure 12: Scheme of the hysteretic behaviour adopted for the materials.

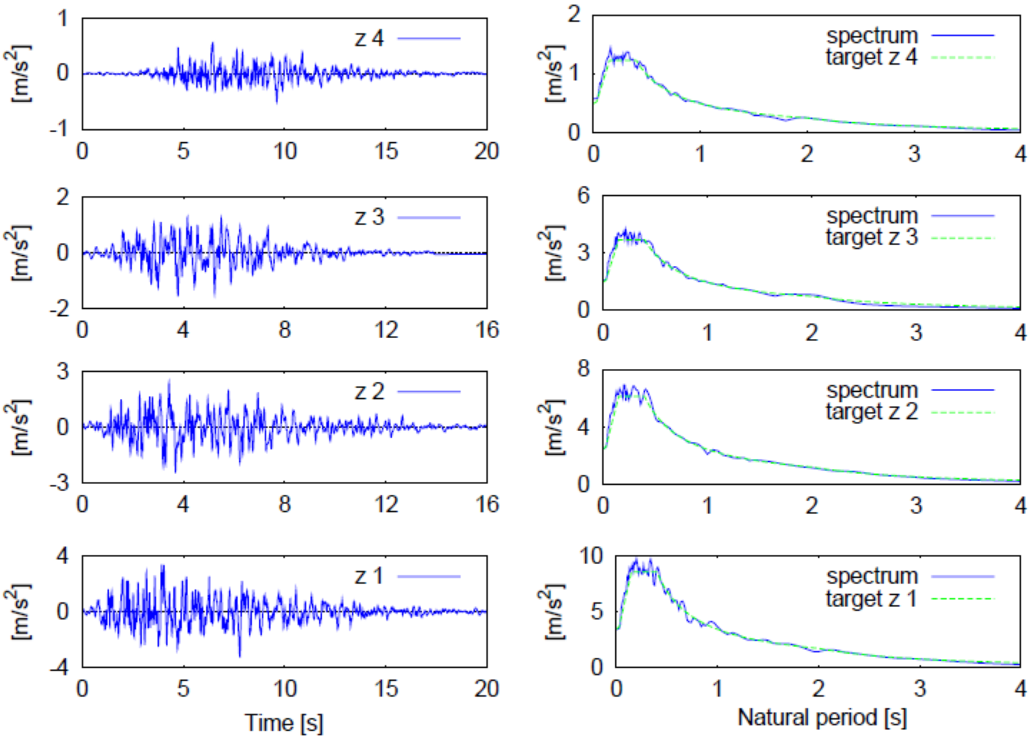


Figure 13. Artificial accelerograms used in the non-linear dynamic analyses, and corresponding spectra compared with those of the seismic zones 1, 2, 3, 4.

With regard to the seismic action, only the horizontal component was considered. In theory, the vertical component interacts with the horizontal response, but for this case, the differences between bending and axial vibration modes - in terms of eigenvalues – were relevant, and considered thence to be negligible. A set of artificially generated accelerograms was considered, as shown in Figure 13.

4.3 Results of the non-linear dynamic analyses

After calibrating the plane models in the elastic field by means of modal analyses and comparisons with the results of the full 3D model, non-linear dynamic analyses of the three models have been performed under accelerograms corresponding, respectively, to the 4 macro-seismic zones of the Italian territory (characterized by a growing hazard from 4 to 1).

In the following, the results obtained for the models in terms of time history of the horizontal displacements (calculated at the level of the top floor), cumulated hysteretic and kinetic energy, S_{22} stress component and final deformed shape are presented and discussed.

Plane Model "Section 1"

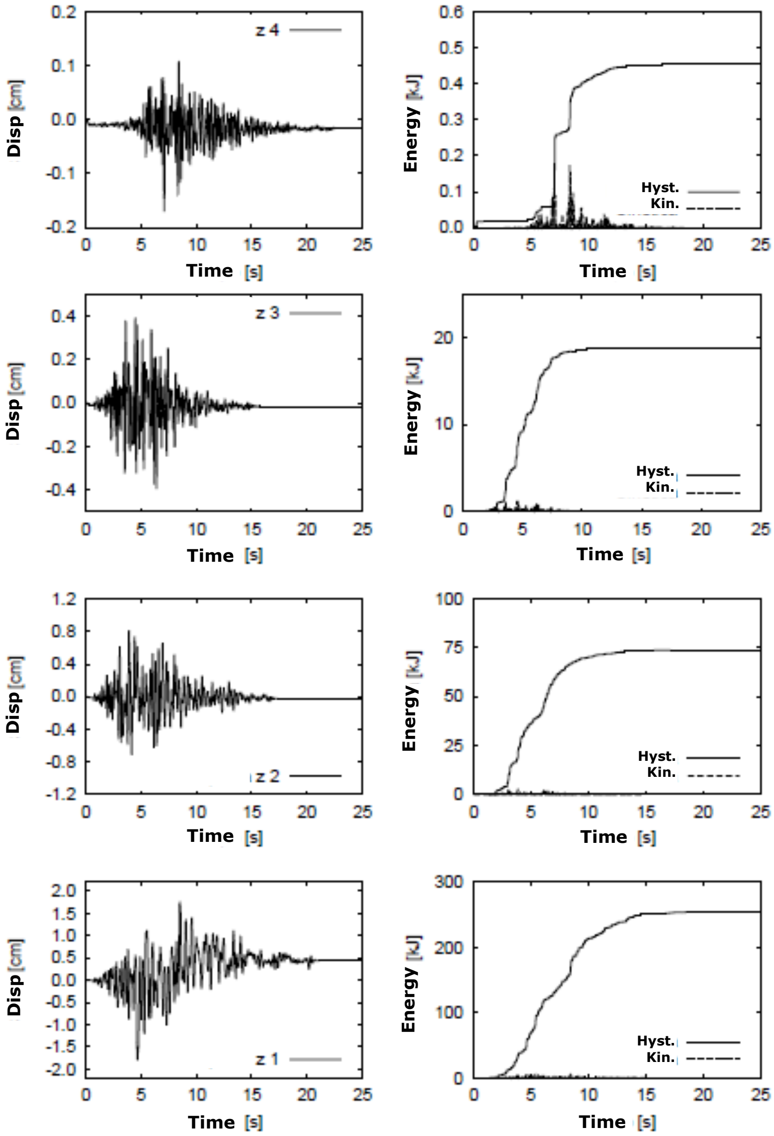


Figure 14. Plane model Section "1": Displacement time history, kinetic and hysteretic energy.

Figure 14 shows the time history of the horizontal displacements calculated at the level of the top floor, and the cumulated hysteretic and kinetic energy. It can be noted that, in the case of the most severe accelerogram (Z1), the maximum horizontal displacement is 2 cm. This value can be considered incompatible with the safety of this type of buildings. Moreover, at the end of the time history, a permanent displacement of approximately 5 mm is attained.

Also under the accelerogram corresponding to the Zone 2 (Z2), the maximum displacement is quite high (1 cm). Looking at the diagram of the hysteretic dissipation, it can be noted that also in this case important plastic dissipation occurs, and for masonry structures this implies a significant irreversible damage.

Results in terms of maps of S_{22} stress component and the deformed shape at the end of the loading history is shown in Figure 15 for the accelerograms Z1, Z2, Z3, Z4.

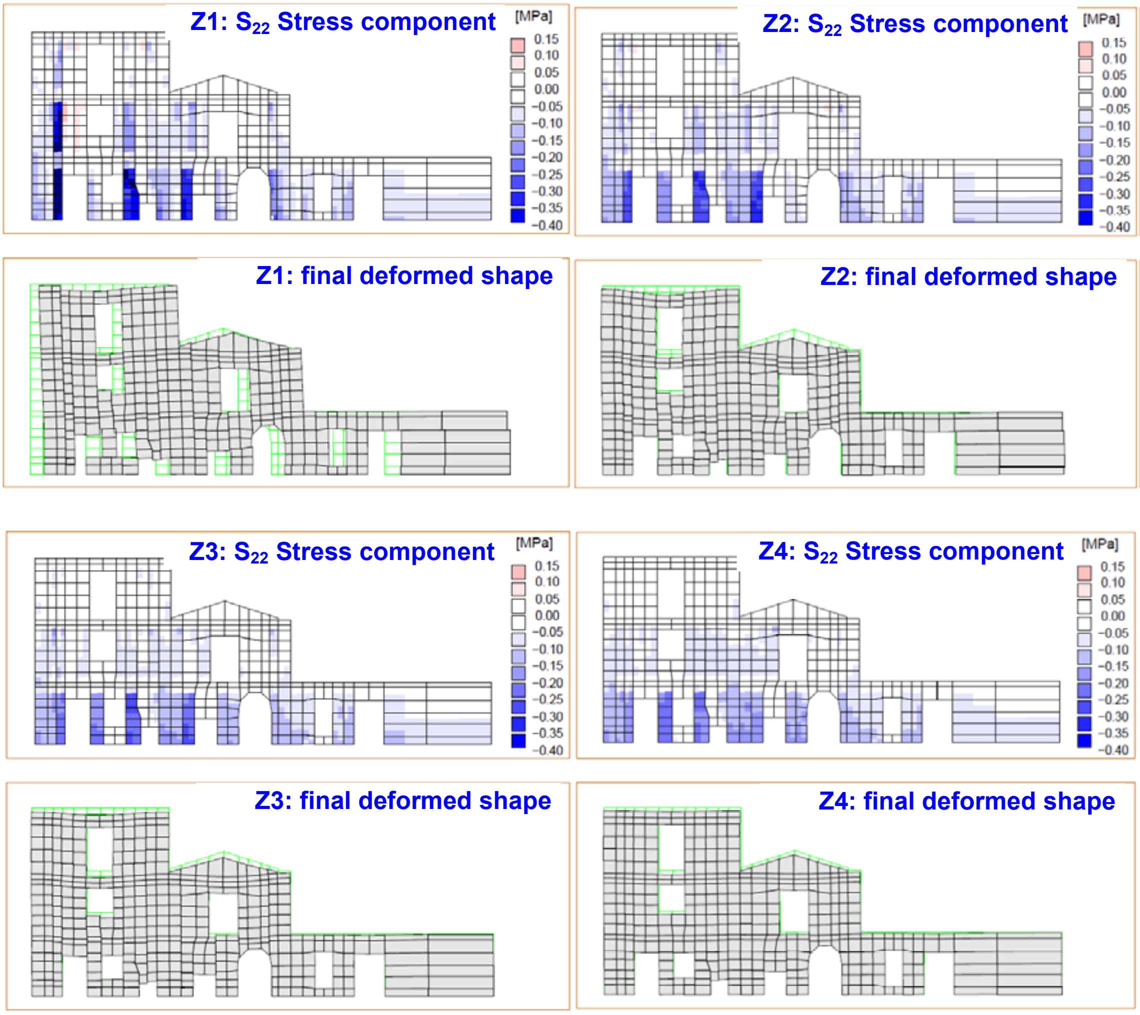


Figure 15. Plane model Section “1”: Final deformed shape and S22 stress component for the 4 accelerograms.

The accelerogram Z4 does not produce significant damage, since the trend of the distribution of S_{22} stress component is substantially the same than that under gravity loads. This is also confirmed by the final deformed shape.

In the results of the accelerogram Z3, some significant damage can be observed, predictable also based on the hysteretic energy dissipation (Fig. 14). More in detail, the map of the S_{22} stress component shows a peak in the central piers of the building, with values of the order of 0.3 MPa. The damage mechanism can be deduced from the observation of the final deformed shape. The central wall piers, between the openings of the second level shows a vertical shift. Overall, this framework can be classified as slight damage.

The accelerogram Z2 produces a medium/high level of damage, with relevant plastic phenomena, as highlighted by the diagram of the hysteretic energy (Fig. 14). The map of the S_{22} stress component is similar to the previous case, with a peak in the central piers of the building, with values of the order of 0.35 MPa. Also, the damage mechanism is similar to the previous one. Overall, this framework can be classified as medium-high damage, with the possibility of local collapse mechanisms in the central area of the aggregate. The analysis of the damage patterns previously described points out that a strong vulnerability factor is related to the misalignment of the openings.

The accelerogram Z1 produces very severe damages, with a probable collapse of the entire building. The high values of hysteretic energy highlighted in Figure 14 are very well explained by the presence of relative translations between entire blocks, both horizontal and vertical. The map of S_{22} stress component is completed altered with respect to the behavior under gravity loads. Basically, this means that the structure, at the end of the earthquake, has no residual safety margin with respect to vertical actions. The mechanism of damage is different from the previous ones, because it shows a strong shear component, both along horizontal cracks at the first level, and along vertical fractures that completely crushes the masonry walls into separate pieces. Overall, this framework can be classified as a complete collapse.

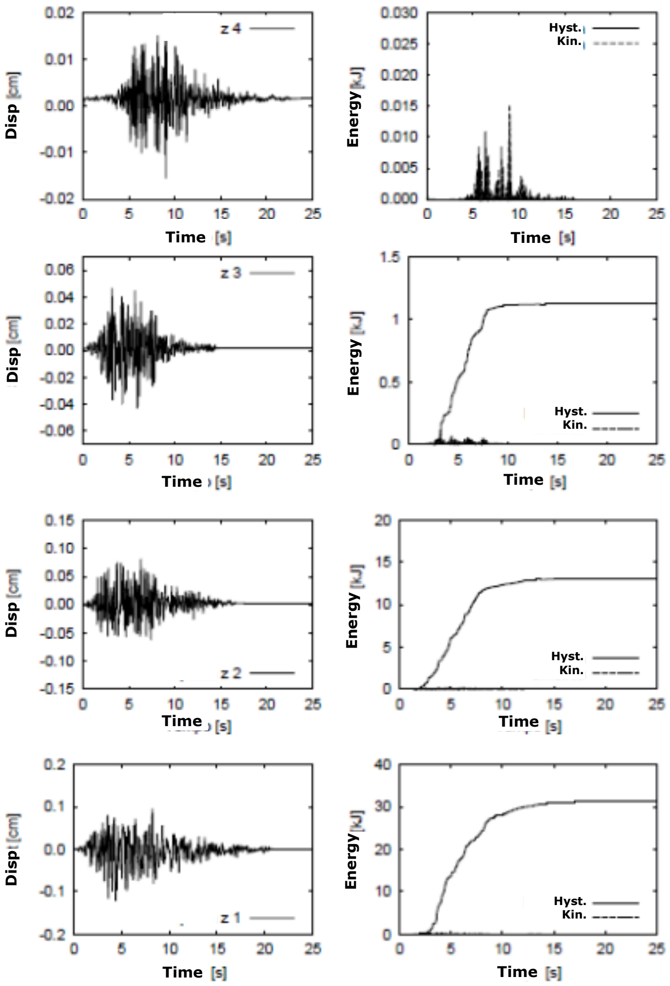


Figure 16. Plane model Section “3”: Displacement time history, kinetic and hysteretic energy.

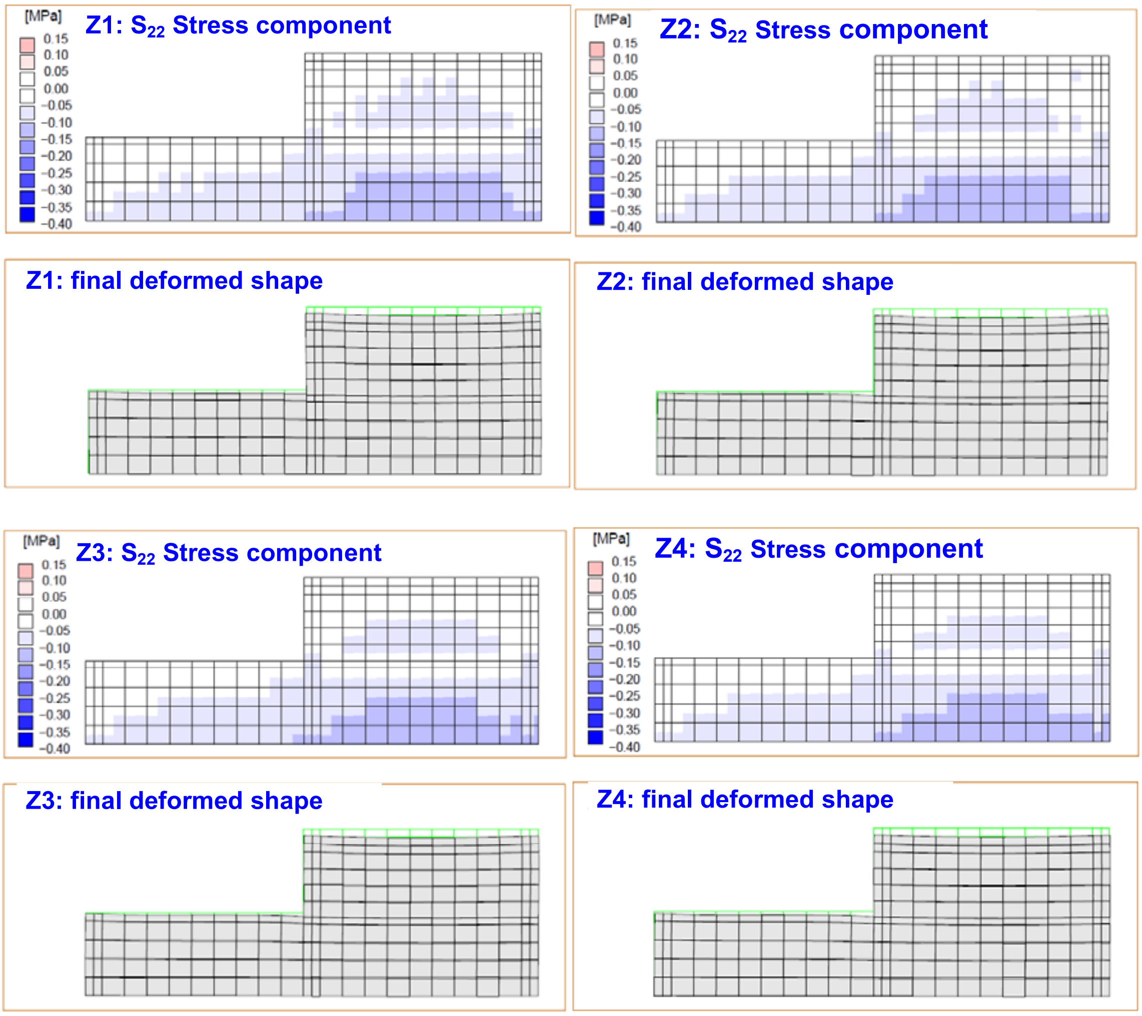


Figure 17. Plane model Section "3": Final deformed shape and S22 stress component for the 4 accelerograms.

Plane Model "Section 2"

The results obtained for the second plane model are quite similar to those of Section 1, and will not be here discussed in detail, for brevity sake.

Plane Model "Section 3"

Under the accelerogram Z4, no damage is observed, as shown by the distribution of S_{22} stress component, which is substantially the same than that under gravity loads. This is also confirmed by the final deformed shape. Also, the accelerogram Z3 does not produce significant damage, as it can also be seen by the plot of the dissipated hysteretic energy, which is very low (Fig. 16). The map of the S_{22} stress component, like in the previous case, is substantially the same than that under gravity loads. No significant damage mechanism is observed and, overall, this case can be classified as almost negligible damage. The accelerogram Z2 produces very low level of damage, with hardly appreciable plastic phenomena, as also as highlighted by the diagram of the hysteretic energy (Fig. 16). Overall, this case can be classified as very low damage level.

Even the accelerogram Z1 induces low damages. However, in this case the values of the dissipated hysteretic energy are higher than those attained in the previous cases, but anyway there is no a significant degradation of the overall structural behavior. Overall, this situation can be classified as low damage level, but only in consideration of the presence of some dissipated energy.

5 CONCLUSIONS

The objective of the research study was to make a comparison between the results of 1st level vulnerability assessment and a more detailed analysis based on a mechanical modeling performed by different approaches.

For the analyses a reference aggregate building has been considered. Its morpho-typological and mechanical properties were defined based on the survey and analysis of the residential buildings present in the historical center of Foggia ("Quartiere Borgo Croci"). This part of the city dates to the 18th century, when the city was reconstructed after the disastrous earthquake occurred in 1731, and buildings were constructed according to basic anti-seismic criteria: regularity in plan and elevation, resistant masonry walls, reduced slenderness of the walls. Subsequent historical events strongly altered the original configuration, leading to the present situation, which is extremely irregular and badly organized, mainly because of the proliferation of super-elevations, poorly designed and executed.

The methodology adopted in the research study consisted in performing an initial study at the scale of the entire building aggregate, and then focus to a more detailed scale, in which three plane models were defined, corresponding to the two end sections and to an intermediate section. The numerical analyses in dynamic non-linear field have shown significant differences in the vulnerability between the different parts of the aggregate.

"Section 1" and "Section 2" exhibit total collapses under the accelerogram Z1, partial collapses under the accelerogram Z2, and substantial damage under the accelerogram Z3. On the other hand, the mechanical model that refers to the "Section 3" (corresponding to a structural unit located in a central position) does not show significant damage, except than under the accelerogram Z1, but also in this case damages are very limited.

AKNOWLEDGEMENTS

The authors gratefully acnowledge the financial support of AdB Puglia - Autorità di Bacino della Regione Puglia (Basin Authority of Puglia), within the program Studio di Fattibilità per il Monitoraggio e la Messa in Sicurezza delle Aree Urbane a Rischio di Stabilità Statica e Vulnerabilità Strutturale nella Città e Provincia di Foggia (CIPE 20/2004), and the local governments Province of Foggia and City of Foggia.

REFERENCES

[1] I. Senaldi, G. Magenes, A. Penna, Numerical investigations on the seismic response of masonry building aggregates, Advanced Materials Research 133-134, 715-720, 2010.

[2] G. Uva, C.A. Sanjust, S. Casolo, M. Mezzina, ANTAEUS project for the regional vulnerability assessment of the current building stock in historical centers *International Journal of Architectural Heritage, 10*(1), pp. 20-43, 2016. http://dx.doi.org/10.1080/15583058.2014.935983

[3] S. Casolo, Modelling in-plane micro-structure of masonry walls by rigid elements. *International Journal of Solids and Structures, 41*(13), pp. 3625-3641, 2004. http://dx.doi.org/10.1016/j.ijsolstr.2004.02.002

[4] S. Casolo, Macroscale modelling of microstructure damage evolution by a Rigid Body and Spring Model, *Journal of Mechanics of Materials and Structures* 4 (3), pp. 551-570, 2009. http://dx.doi.org/10.2140/jomms.2009.4.551

[5] ABAQUS. The Abaqus Software is a product of Dassault Systèmes Simulia Corp., Providence, RI, USA. © Dassault Systèmes, 2010.

[6] S. Casolo, F. Pena, Rigid element model for in-plane dynamics of masonry walls considering hysteretic behaviour and damage, *Earthquake Engineering and Structural Dynamics* 36 (8), pp.1029-1048, 2007. http://dx.doi.org/10.1002/eqe.670

[7] S. Casolo, Macroscopic modelling of structured materials: relationship between orthotropic Cosserat continuum and rigid elements. *International Journal of Solids and Structures*, 43 (3-4), 2006, pp.475-496. http://dx.doi.org/10.1016/j.ijsolstr.2005.03.037

[8] *Survey of monumental buildings located in national or regional natural parks*. Tech. rep., GNDT - (Italian Group for the defense against earthquakes), Italian Ministry of Labour, Civil Protection, Rome, Italy (2001). (In Italian).

[9] Corsanego, A.: Sismic risk assessment at the urban and regional scale in Italy: some examples. *Annali di Geofisica* 36(1), 211–218 (1993). URL http://www.earth-prints.org/bitstream/2122/3612/1/18

[10] Zuccaro, G.: Metodi di Analisi di Vulnerabilità e Matrici di Probabilità del Danno. In: *La Meccanica delle Murature tra Teoria e Progetto*. Messina, Italy (1996).

[11] Dolce, M., Masi, A., Marino, M., Vona, M.: Earthquake Damage Scenarios of the Building Stock of Potenza (Southern Italy) Including Site Effects. *Bulletin of Earthquake Engineering* 1(1), 115 (2003). http://dx.doi.org/10.1023/A:1024809511362

[12] Lagomarsino, S., Giovinazzi, S.: Macroseismic and mechanical models for the vulnerability and damage assessment of current buildings. *Bulletin of Earthquake Engineering* 4(4), 415 (2006). http://dx.doi.org/10.1007/s10518-006-9024-z

[13] Rota, M., Penna, A., Strobbia, C., Magenes, G.: Typological seismic risk maps for Italy. *Earthquake Spectra* 27(3), 907–926 (2011). http://dx.doi.org/10.1193/1.3609850

[14] Casolo, S., Neumair, S., Parisi, M.A., Petrini, V.: Analysis of seismic damage patterns in old masonry church facades. *Earthquake Spectra* 16(4), 757–773 (2000). http://dx.doi.org/10.1193/1.1586138

[15] S. Casolo, G. Uva. Nonlinear analysis of out-of-plane masonry façades: Full dynamic versus pushover methods by rigid body and spring model. *Earthquake Engineering and Structural Dynamics, 42*(4), pp.499-521, 2013. http://dx.doi.org/10.1002/eqe.2224

[16] S. Lagomarsino, S. Podestà. Seismic vulnerability of ancient churches: I. Damage assessment and emergency planning. *Earthquake Spectra* 20(2), 2004, pp. 377-394.

[17] S. Lagomarsino, S. Podestà. Seismic vulnerability of ancient churches: II. Statistical analysis of surveyed data and methods for risk analysis. *Earthquake Spectra* 20(2), 2004, pp. 395-412.

[18] Casolo, S., Diana, V., & Uva, G. (2016). Influence of soil deformability on the seismic response of a masonry tower. *Bulletin of Earthquake Engineering*, 2016 http://dx.doi.org/10.1007/s10518-016-0061-y

[19] S. Casolo, C.A. Sanjust. Seismic analysis and strengthening design of a masonry monument by a rigid body spring model: The "Maniace Castle" of Syracuse. *Engineering Structures* 31(7), 2009, pp.1447-1459. http://dx.doi.org/10.1016/j.engstruct.2009.02.030

[20] S. Casolo, G. Milani, C.A. Sanjust, A. Taliercio. Maniace Castle in Syracuse, Italy: Comparison between present structural situation and hypothetical original configuration by means of full 3D FE models. *Open Civil Engineering Journal*, 6(SPEC.ISS.1), 2012, pp. 173-187. http://dx.doi.org/10.2174/1874149501206010173

[21] G. Milani, S. Casolo, A. Naliato, A. Tralli. Seismic Assessment of a Medieval Masonry Tower in Northern Italy by Limit, Nonlinear Static and Full Dynamic Analyses. *International Journal of Architectural Heritage* 6(5), 2011, pp.489-524. http://dx.doi.org/10.1080/15583058.2011.588987

[22] S. Casolo, G. Milani, G. Uva, C. Alessandri, Comparative seismic vulnerability analysis on ten masonry towers in the coastal Po Valley in Italy, *Engineering Structures*, 49, 2013, pp.465- 490. http://dx.doi.org/10.1016/j.engstruct.2012.11.033

[23] S. Casolo. Rigid element model for non-linear analysis of masonry façades subjected to out-of-plane loading. *Communications in Numerical Methods in Engineering*, 15(7), 1999, pp.457-468.

[24] S. Casolo. Modelling the out-of-plane seismic behaviour of masonry walls by rigid elements. *Earthquake Engineering and Structural Dynamics, 29*(12), 2000, pp.1797-1813. http://dx.doi.org/10.1002/1096-9845(200012)29:12<1797:AID-EQE987>3.0.CO;2-D

[25] S. Casolo, G. Uva. Out-of-plane seismic response of masonry façades some comparisons among full dynamic and pushover analysis. [Risposta sismica fuori del piano di facciate in muratura: Confronto tra analisi dinamiche non-lineari e analisi pushover], *Ingegneria Sismica*, 27(3), 2010, pp. 33-54, ISSN: 03931420.

[26] S. Casolo, G. Milani. A simplified homogenization-discrete element model for the non-linear static analysis of masonry walls out-of-plane loaded. *Engineering Structures, 32*(8), 2010, pp.2352-2366. http://dx.doi.org/10.1016/j.engstruct.2010.04.010

[27] S. Casolo, G. Milani. Simplified out-of-plane modelling of three-leaf masonry walls accounting for the material texture. *Construction and Building Materials, 40*, 2013, pp.330-351. http://dx.doi.org/10.1016/j.conbuildmat.2012.09.090

[28] A. Fiore, F. Porco, D. Raffaele, G. Uva, About the influence of the infill panels over the collapse mechanisms actived under pushover analyses: two case studies, Soil Dynamics and Earthquake Engineering, 39, 2012, pp.11-22. http://dx.doi.org/10.1016/j.soildyn.2012.02.004

[29] G. Uva, F. Porco, D. Raffaele, A. Fiore, On the role of equivalent strut models in the seismic assessment of infilled RC buildings, Engineering Structures, 42, 2012, pp.83-94, http://dx.doi.org/10.1016/j.engstruct.2012.04.005

[30] P.B. Lourenço. Computations on historic masonry structures. *Progress in Structural Engineering and Materials* 4(3), 2002, pp.301-319.

[31] S. Casolo, A. Fiore, F. Porco, D. Raffaele, C.A. Sanjust, G. Uva. A multi-level approach for the numerical modelling of complex monumental buildings: Seismic assessment of the "maniace castle" of syracuse. In: *Civil and environmental engineering: Concepts, methodologies, tools, and applications* (pp. 1352-1379). http://dx.doi.org/10.4018/978-1-4666-9619-8.ch061