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Florence: seismic assessment of not-historical masonry buildings population

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Abstract. *The city of Florence, whose historic center has been declared a World Heritage Site by UNESCO in 1982, is known all over the world for its historical and artistic heritage. However, more than half of its buildings population was made in the XX century, therefore presenting the typical problems and weaknesses proper of pre-normative "recent" constructions. The XX century buildings population is made both by RC and masonry constructions. This work deals with the seismic assessment of masonry buildings made in Florence after the II world war. In those decades, in fact, a large number of masonry buildings has been made, both as private and public projects. The public interventions, belonging to housing national projects, were aimed at providing homes for needing citizens; they cover a large number of buildings, all over the town, having similar features and material. In the paper, a brief description of the development of the public housing in Florence has been presented, and some building—types have been assumed to represent the public buildings population. One of this buildings-type has been selected for an evaluation in terms of seismic assessment. To this purpose two case-studies, belonging to the same type, have been considered, and different assumptions, all consistent to the collected information, have been made about their masonry strength, elastic stiffness and floor weight, in order to represent a variety of possible situations compatible with the proposed classification. The seismic assessment of the case-studies has been made by performing a nonlinear static analysis, where the seismic input has been described according to the Italian current Technical Code. The obtained results evidence a safety level lower than expected for new constructions, whose acceptability depends on the model assumptions.*

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1 INTRODUCTION

The city of Florence is known all over the world as representative of the Italian architecture, and symbol of Renaissance. However, more than half of its residential buildings population was made in the XX century, before the support of the recent technical seismic legislation. These buildings, secondary for importance but primary for number, are still in use, even if their safety level has never been checked.

By comparing the constructive activity in Florence to the one of the entire Country (Figure 1), it can be noted that in Florence the construction activity is gathered mostly in the beginning of the century, and it is made mostly by masonry buildings. Pre-normative masonry buildings, therefore, represent the large majority of the residential reality of the town.

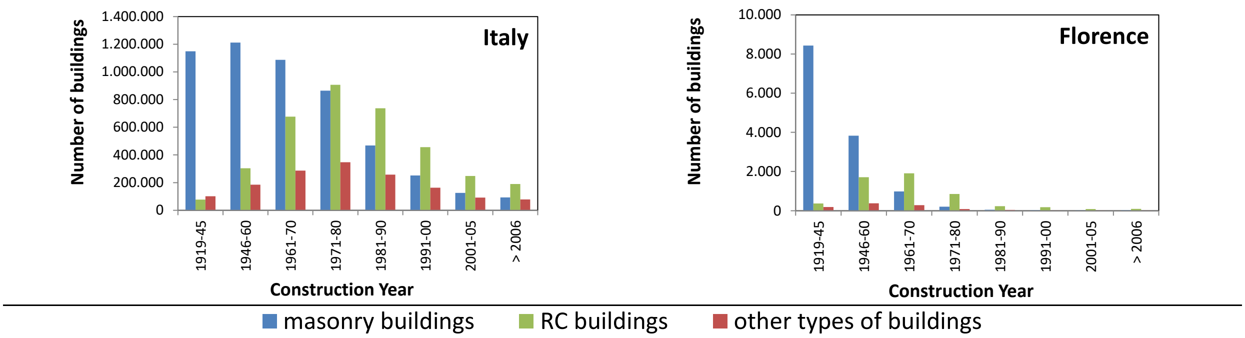


Figure 1. Construction activity in Italy and in Florence (ISTAT).

In the XX century many public housing interventions have been made in Florence, as well as in all over the country. They have been very important, since they comprehend a meaningful number of constructions, and their projects are, usually, more detailed and better preserved than the others. Furthermore, the buildings belonging to the public housing are more "homogenized" than the private ones, and they have settled a standard in morphology, mechanical properties and – consequently – performance; therefore they can be considered important in representing the XX century houses in Florence.

Even as regards the public housing interventions, the masonry buildings were much more numerous than the RC ones, as can be seen in Figure 2.

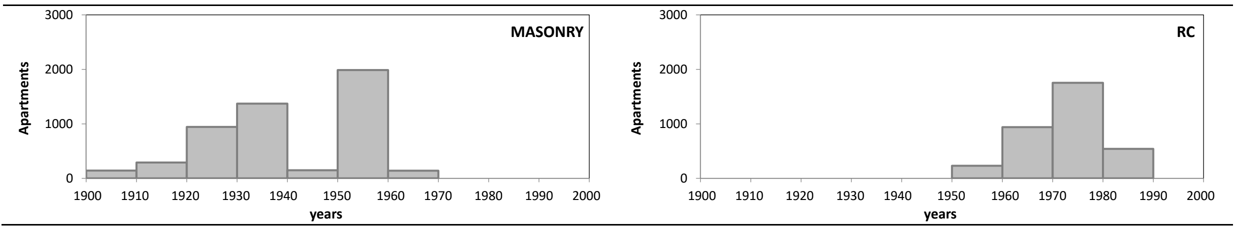


Figure 2. Public housing activity in the XX century.

This paper is focused on the evaluation of the seismic assessment of standard masonry buildings belonging to the public housing interventions made in Florence in the XX century. The study refers to two case-studies, assumed to represent a typical masonry building. The selection of the case-studies has been made after a careful survey on the buildings population belonging to the public housing interventions in Florence in the XX century. In the paper, a brief description of the development of the public housing in Florence has been presented (Section 2), in order to better frame the quantity and the properties of these residential buildings. In Section 3 the masonry main building types constituting the housing interventions in

Florence have been briefly presented, and one of them has been selected for the seismic assessment. Two different case-studies, differing for storey number and main walls disposition, have been selected and described. In Section 4 the seismic assessment of the case-studies, based on a nonlinear static analysis, has been presented and compared to the current safety standards.

2 MAIN HOUSING INTERVENTIONS IN FLORENCE

The first public housing intervention, in Florence, go back to 1849, when the SAE (anonymous housing society) built a house for workmen [1]. In the following years (1865-71) Florence became the Capital of Italy, and the housing polity was consequently affected by politic needs, like pursuing representativeness and security by replacing many old and tiny buildings with new middle-class constructions.

With the premature relocation of the Capital, Florence experienced a period of decline, and the city center become the refuge of the poorest part of the population. Many other anonymous interventions succeeded, to help the poorest population. One of the most representative was the housing committee “Comitato per le Case ad uso degli Indigenti”, founded in 1855, which made 342 apartments (black circles in Figure 3) between 1886 and 1916.

At the beginning of the XX century, with the increasing of the industrial activity and the life standards, many new buildings, both private and public, were made. In 1903, with the first Code on the public housing [2], the public administration begin to make housing interventions, starting a process which lead, in 1909, to the institution of the IACP “Ente Autonomo per le Case Popolari” [1]. The first intervention consisted of 223 apartments, having good quality standards, according to the time (such as the supply of electricity and potable water). Such buildings had limited dimensions, and they were located in the suburban – and more inexpensive - areas. These constructions were very similar to the private houses, both for the plan organization and for the decorative choices. In the meantime, many private apartments were build (490 luxury, 3234 standard and 2212 working-class, respectively), usually without any order or urbanistic criterion. In order to limit the urbanistic disarrangement, a new plan was introduced in 1908, whose effects were compromised by the IWW.

After the war the housing activity restarted with the construction of new buildings in the suburban areas, whilst the central areas were left for middle-class housing and commercial use. Between 1922 and 1943, with the Fascist regime the housing emergency was faced by the “Istituto Autonomo Fascista Case Popolari”, which made several buildings, with intensive occupational density and low standards. In 1935 the local Institutions merged in the National housing association (Consorzio Nazionale fra gli Istituti Autonomi per le Case Popolari), and new housing standards were set: higher buildings (4 storeys), smaller dimensions of rooms and a different plan-distribution. In the late ’30 the quality of the apartments made for social housing became lower, together with the regime decline. Special attention should be paid to the house made for the soldiers families [1], which had higher standard quality and dimensions (6-10 rooms each) and presented a special arrangement, with buildings having a train-like aggregation.

After the IIWW the public housing had a further development. The tough economic situation generated a housing impasse, and the Office in charge was not able to face the situation; therefore the reconstruction projects were mostly private. In this situation, the birth of “INA-CASA” [3] represented a turning point in the public housing. INA-CASA was the building branch of the National Insurance Company (INA) [2]. Its aim was to construct houses to assign to the working class, adopting a ransom policy. INA-CASA cooperated with local companies: INPS and social security companies were in charge of the funding, the contractors found the area for constructions, whilst IACP managed the buildings administration. In the years 1950-52 INACASA issued a framing regulation for the project of the first step of interventions (planned for the first 7 years of activity), in order to have homogeneous buildings all over the Country. The model was inspired to the Scandinavian experiences [1], with comprehensive quartiers, having all the devices and infrastructures for the inhabitants social and cultural life. A minimum surface was fixed for any apartment function, and few layouts were selected for all the interventions. The hallways were minimized to assure a minimum surface to the main rooms, and cloisters and courtyards were eliminated. Moreover, strict limits are assumed for the construction costs, with the effect of an extensive use of poor technology.

Due to the poor materials, the houses experienced a fast decline. Therefore, the next generation of guidelines (made in 1956-57 [4]) paid more attention to the quality and even the urban planning, the common areas, and the design coordination were improved. Starting from 1963, with the replacement of INA-CASA by GESCAL (Gestione Case per i Lavoratori), many further interventions were made in Florence, and many other actors, such as ATER and ARDSU, appeared in the scenario. Namely, ATER [2] managed the construction of around 12000 apartments, mainly still in use. The city itself built around 4000 units, while different – sometimes private - companies made around 6000 units.

As a results of the whole public housing activity occurred from 1849 to nowadays, more than 22000 apartments, corresponding to almost 15% of the total building population, have been made. Figure 3 shows the main interventions of public housing made in Florence. They have been classified after their years of construction, dimension and material (masonry or reinforced concrete, RC), besides the location inside the town area.

Age of the interventions	Structural material

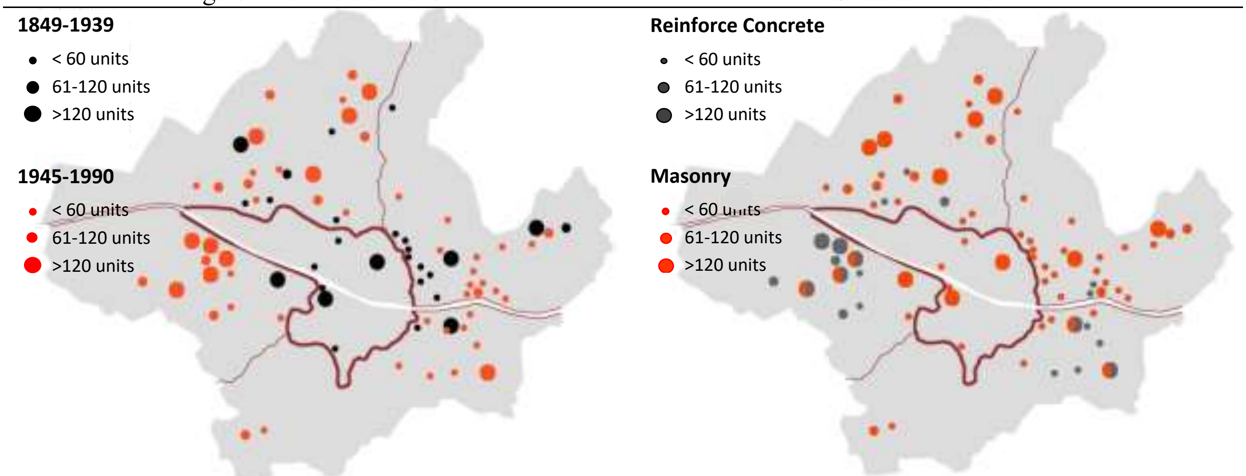


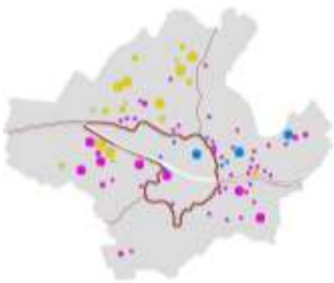
Figure 3. Main interventions of public housing made in Florence.

3 THE SELECTED CASE-STUDIES

3.1 Main types of buildings belonging to the public housing

As evidenced in Section 2, the buildings belonging to the public housing interventions present different properties, in terms of general features and mechanical properties, above all depending on age of construction, and on the general frame (technical, political and normative) of the time. The classification has been limited to the aggregation properties of masonry buildings, since the RC ones are beyond the scope of this paper. Table 1 shows the main information about the proposed classification, made after the aggregation geometry of the buildings. In this work the Type_1 has been selected for the seismic assessment.

Table 1. Main building-types made in the public housing masonry interventions

	Buildings Types	Storey number	Construction years
	Type_1: BLOCK	2-4	late '800, 30s-today
	Type_2: LINE	3-6	1870-1936, 30s-today
	Type_3: ANGLE	3-6	1800, 30s

3.2 The considered building-type

The building-type selected for analysis is the one indicated as Type_1 in Table 1. The Type_1, together with the Type_2, is the most diffuse in Florence, and it can be found in different areas (see Table 1) and with different features. The peculiarities are the independency of each block, the rectangular plan with the stairs in the middle, which serve two apartments for each storey. The foundation is usually made by masonry walls, having the same geometry and larger depth than the upper ones. The hip roof is sustained by a proper perimeter concrete kerb.

In Florence many public intervention have adopted this type. Therefore, the buildings assumed to belong to this type can differ each other for material quality, walls disposition, floors orientation, number and type of storeys, etc. In this work, therefore, two different buildings, both belonging to the Type_1, have been considered.

3.3 The considered case-studies

Two different case-studies have been considered in this work, both classified as Type_1 and belonging to the same intervention, made between 1949 and 1951.

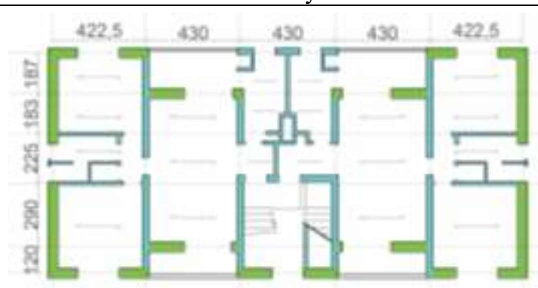
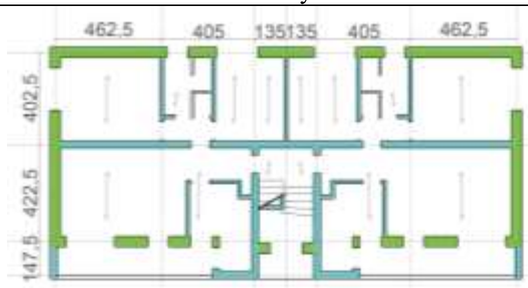
Case-study #1	Case-study #2
	
Masonry A	Masonry B
Number of storeys: 4 Masonry walls: foundation: 55cm (A) 1st, levels: 45 cm (A), 30 and 15 cm (B) 2nd, 3rd, 4th levels: 30 and 15 cm (B) Main walls: *Y*-direction	Number of storeys: 3 Masonry walls: foundation: 50 cm (A) 1st level: 45 cm (A), 30 and 15 cm (B) 2nd and 3rd levels: 30 and 15 cm (B) Main walls: *X*-direction

Figure 4. The two case-studies: plans and information.

An extensive information about the original projects has been collected, and the representation of the buildings has been made consequently. In Figure 4 the plans of the two case-studies are shown, together with the main assumptions made for the analysis. The two case-studies differ each other mainly for the direction of the main masonry walls and for the number of storeys. In Fig. 4 two different masonry types have been evidenced, respectively made of coursed rubble masonry (A) and plain brick one (B). The coursed rubble masonry is made of stones of various size and by horizontal courses, 1 meter spaced, made of bricks. The plain brick masonry, instead, is made with lime mortar.

The floors are made of RC joists alternated to hollow bricks, topped by a 4-cm concrete slab, and they have a thickness of 20 cm; the top storey floors, where there are not live loads, have the same technology but lower (12/16 cm) depth.

4 THE SEISMIC ANALYSIS

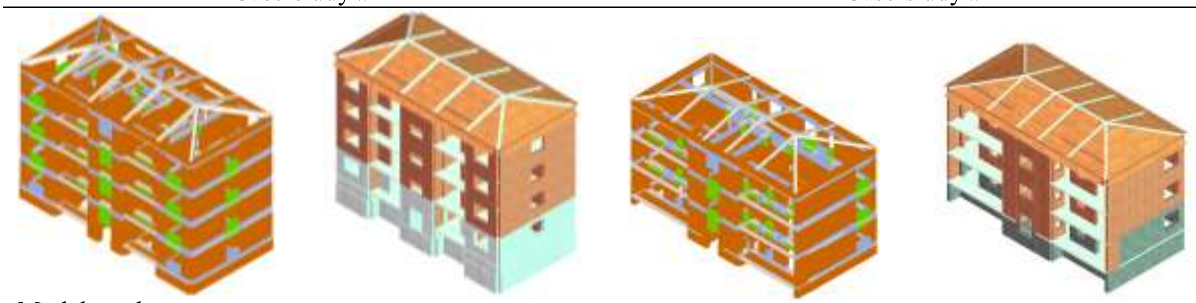
The seismic assessment has been made by performing a nonlinear static analysis, comparing the structural capacity of the case-study under horizontal loading to the seismic demand defined after the seismic classification of the site. In Section 4.1 the numerical models adopted for analysis are described, while in Sections 4.2 and 4.3 the capacity and the seismic demand of the case-study are presented, respectively. Finally Section 4.4 shows the obtained seismic assessment of the case-studies.

4.1 The models

The structural model has been made through the software 3Muri [5], which is the commercial version of the Tremuri computer code developed by [6]. The masonry have been represented as plane panels with a nonlinear beam-behavior [7]. A no-tension bilinear relationship has been assumed for the masonry, with a nonlinear reallocation of the compressive stress based on the stress-block model. The floors have been modelled through plane stiffness membranes having an equivalent thickness, two orthogonal elasticity moduli and an equivalent shear modulus. Therefore, the influence of the floor deformability has been considered in the global response of the structural system.

In Figure 5 the 3D views of the numerical models can be seen, while Tables 2 and 3 show the main values assumed for the mechanical properties of masonry, concrete and steel, respectively.

Case-study #1 | Case-study #2



Model mesh | Model texture | Model mesh | Model texture

Figure 5. 3D views of the structural model.

The mechanical properties of masonry have been assumed according to the current Italian Technical Code [8], which provides two different values (minimum and maximum) for each quantities. The value assumed for the masonry A refer to a "improvement coefficient" equal to 1.1, according to the instructions provided by the Code [8]. Furthermore, different safety factors (Confidence Factors, *CF*) can be assumed depending on the achieved knowledge level of the building. The masonry stress, therefore, has been described by the alternative values, respectively found as the maximum of the Code range, related to *CF*=1.00, and the minimum, related to *CF*=1.35.

Table 2. Mechanical quantities assumed for the masonry [8, C8A.2].

	Masonry_A				Masonry_B			
	symbol	unit	min value	max value	symbol	unit	min value	max value
Normal stress	$f_{m,A}$	N/cm^2	260	380	$f_{m,B}$	N/cm^2	240	400
Shear stress	$\tau_{0,A}$	N/cm^2	5.6	7.4	$\tau_{0,B}$	N/cm^2	6.0	9.2
Young modulus	$E_{m,A}$	N/mm^2	1500	1980	$E_{m,B}$	N/mm^2	1200	1800
Shear modulus	$G_{m,A}$	N/mm^2	500	600	$G_{m,B}$	N/mm^2	400	600
Specific weight	ρ_A	KN/m^3	21		ρ_B	KN/m^3	18	

Table 3. Mechanical quantities assumed for concrete and steel.

	Concrete (class C20/25)			Steel (class "acciaio comune" [Verderame et al. 2011][9])		
quantity	symbol	unit	value	symbol	unit	value
Normal stress	f_c	N/mm^2	24	f_c	N/mm^2	350
Young modulus	E_c	N/mm^2	28600	E_c	N/mm^2	206000

Even the masonry Young modulus has been represented through two alternative assumptions, which are assumed to describe the elastic stiffness of the uncracked (max E_m) and cracked (min E_m) masonry, respectively. Another variable has been introduced through the floors description. Indeed, two different types of floors have been considered, having different weight.

Since no specific information is available about the technology adopted for the floors, two different components, respectively normal and lighten, have been considered. The two floor types present the same structural behavior, changing only in their weight, with the lighten floor about 30% lighter than the other. In Table 4 the assumptions concerning the buildings representation are resumed, and the models adopted in the analysis have been named after them.

Table 4. Models adopted in the analysis.

Model	Code	Masonry strength	Masonry stiffness	Floors weight
Weak and uncracked masonry, regular floors	W_UN_R	minimum	maximum	regular
Weak and cracked masonry, regular floors	W_CR_R	minimum	minimum	regular
Strong and uncracked masonry, regular floors	S_UN_R	maximum	maximum	regular
Strong and cracked masonry, regular floors	S_CR_R	maximum	minimum	regular
Weak and uncracked masonry, lighten floors	W_UN_L	minimum	maximum	lighten
Weak and cracked masonry, lighten floors	W_CR_L	minimum	minimum	lighten
Strong and uncracked masonry, lighten floors	S_UN_L	maximum	maximum	lighten
Strong and cracked masonry, lighten floors	S_CR_L	maximum	minimum	lighten

4.2 The structural capacity

A pushover analysis has been performed on the case-studies, according to the instructions provided by the European [10] and Italian [11] Technical Codes. The adopted horizontal forces have two different patterns, respectively proportional to the mass distribution (*MP*) and to the first proportional mode (*1M*). For each direction of analysis, the forces have been applied - in both ways - to the mass center of each storey (with no eccentricity, *E0*), and to a distance corresponding to an eccentricity of +/- 5% (*E+, E-*), so that six analyses have been performed for each direction and horizontal pattern.

The obtained capacity curves have been represented through bilinear diagrams as required by NTC 2008, by *i)* imposing the areas equalization, and *ii)* assuming that the elastic tract includes the point of the original curve having the 60% of the maximum shear force.

In Figure 6 the pushover families provided by the nonlinear analysis have shown for the model W_UN_R. As can be noted, the buildings evidence a different strength in the two directions; in the case-study #1, which has the main masonry walls along the *Y*-direction, the capacity along the *Y*-direction is almost double than the other one. The case-study #2, instead, has a capacity slightly larger in the *X*-direction. In all cases, the two assumed horizontal patterns play an important role, with the capacity provided by the MP pattern higher and stiffer than the other. The introduced eccentricity, instead, does not affect significantly the obtained capacity. Figure 7 shows the capacity curves obtained for the two case-studies by introducing the considered assumptions about the material and the floors. For sake of simplicity, only the results referred to the cases *MP+E*0 and *1M+E0*, i.e. by considering the horizontal forces along one way only and without any eccentricity, have been shown, to better evidence the effects related to the introduced assumptions.

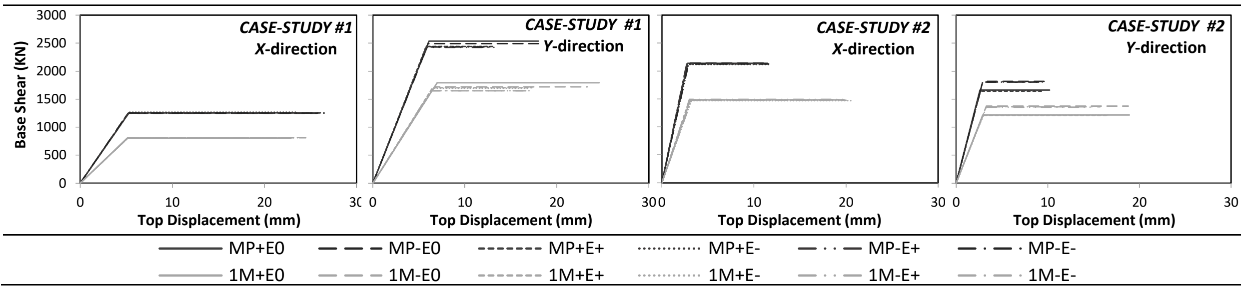


Figure 6. Capacity curves of the case-studies (model W_UN_R).

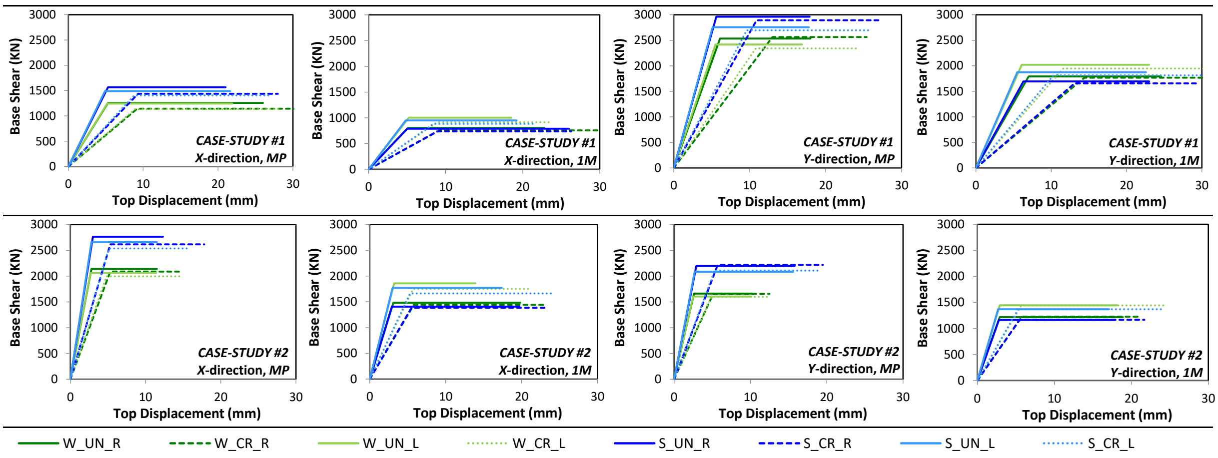


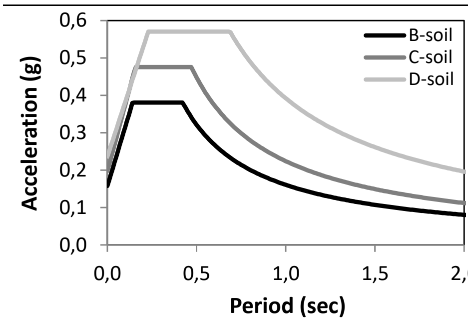
Figure 7. Capacity curves of the case-studies (no eccentricity).

As can be seen from the diagrams, the assumptions concerning the masonry behavior largely affect the obtained results, whilst the floor weight does not seem to play an important role in the seismic capacity of the case-studies.

4.3 The seismic demand

Florence has a PGA of 0,131g for a Return Period, *RP*, equal to 475 years, i.e. for a probability of occurrence equal to 10% in 50 years [12]. The seismic demand has been represented through the seismic spectrum provided by NTC 2008 for the Life Safety (LS) limit state, which is defined as a function of the soil classification, based on the uppermost 30 m shear-wave velocity ($v_{s,30}$). The elastic spectra provided by NTC 2008 slightly differ from the ones provided by EC8; in this work, therefore, the Italian Code has been preferred, since all the available information on the soil are compatible to such classification.

The soil classification of the case-study soil is not easy to achieve. Many investigations [13-15] have been made on the soil of the Florence area, providing a detailed knowledge of the lithostratigraphic subsurface, of the lithoid substrate profile and of the geotechnical properties of its soils. The survey included 1,850 drillings and 32 downhole tests, which provided, for the case-studies vicinity, different strathigraphies, varying in the steadiness of the soil and in the consequent uppermost $v_{s,30}$. As a consequence, different hypotheses regarding the soil classification can be made for area. In this work, therefore, three different hypotheses, all compatible to the available data, have been made, and three elastic spectra (referred to the soil-class B, C and D, respectively) have been considered. Figure 8 shows the elastic spectra considered for the case-studies, together with the information required for the spectra setting.



Parameters for the seismic spectra setting

class	T_R	a_g	F_0	T^*_c	S_T	S_S	Cc	T_C	T_B	T_D
B	475	0.131	2.41	0.30	1.0	1.2	1.397	0.442	0.141	2.215
C	475	0.131	2.41	0.30	1.0	1.5	1.558	0.471	0.157	2.215
D	475	0.131	2.41	0.30	1.0	1.8	2.274	0.687	0.229	2.215

T_R = return period, a_g = ground acceleration, F_0 = amplification factor on the rock-site, T^*_C = beginning period of the velocity-constant branch, S_T = topographic amplification factor, S_S = stratigraphic amplification factor, Cc = soil-type amplification, T_C = period at the beginning of the velocity-constant branch for the assumed soil-type, T_B = period at the beginning of the constant-acceleration branch, T_D = period at the beginning of the displacement constant branch.

Figure 8. Elastic spectra representing the possible seismic input of the case-studies (*RP* = 475 years).

4.4 The seismic assessment

The seismic response of the case-studies has been found by intersecting the elastic seismic spectrum with the capacity curves, represented in the ADRS (Acceleration Displacement Response Spectrum) plane. Such transformation has been made by applying the N2 method [16]. According to Vidic *et al*. [17], the inelastic response has been assumed to coincide to the elastic one for intersection points belonging to the branch of the spectrum with periods over T_C, whilst, otherwise, it is opportunely amplified.

Figure 9 shows the intersection between the spectral capacity and the spectral demand for the two case-studies. As can be noted, the case-study #1 has a lower stiffness then the case-study #2. As a consequence, the case-study #1 intersects the seismic spectrum in the horizontal plateau only for the soil-type D (both directions) and C (*Y*-direction), while in the other cases the intersection occurs for lower ordinates. The case-study #2, instead, is much stiffer, with the effect of intersecting the seismic spectra in the plateau in all cases except for the soil-type D.

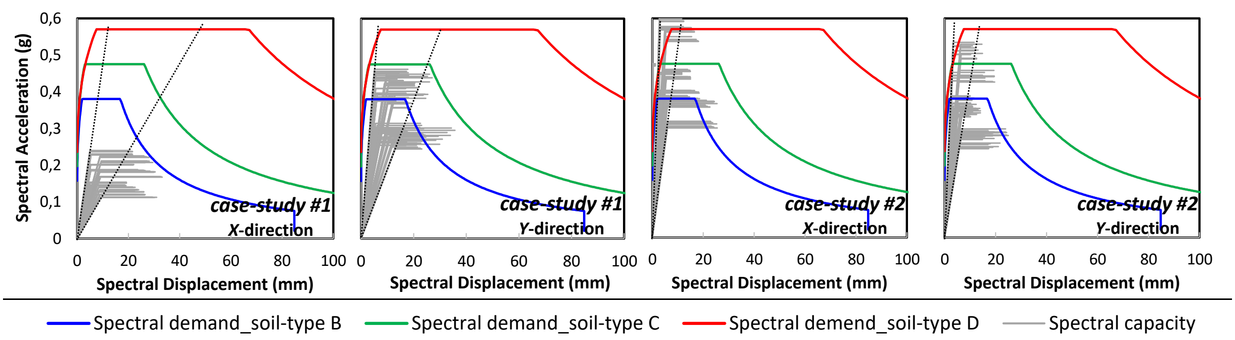


Figure 9. Spectral capacity and demand of the case studies.

The performance of the case-studies, i.e. their accomplishment of the safety level required by NTC 2008 has been expressed through a proper Performance Index (*PI*) as the ratio between the seismic capacity of each model and the corresponding demand. Figure 10 shows the *PI* found for the two case-studies through the adopted models.

As regards the case-study #1, the seismic performance in the *Y*-direction is much better than the one in the *X*-direction. However, when the B-soil is assumed, it results to be verified in almost all cases. At the opposite, *PI* is always lower than unity, i.e. the building does not accomplish the safety requirements, when the D-soil is considered. When the C-soil is assumed, instead, the PI is over the unity for excitations along the *Y*-direction only.

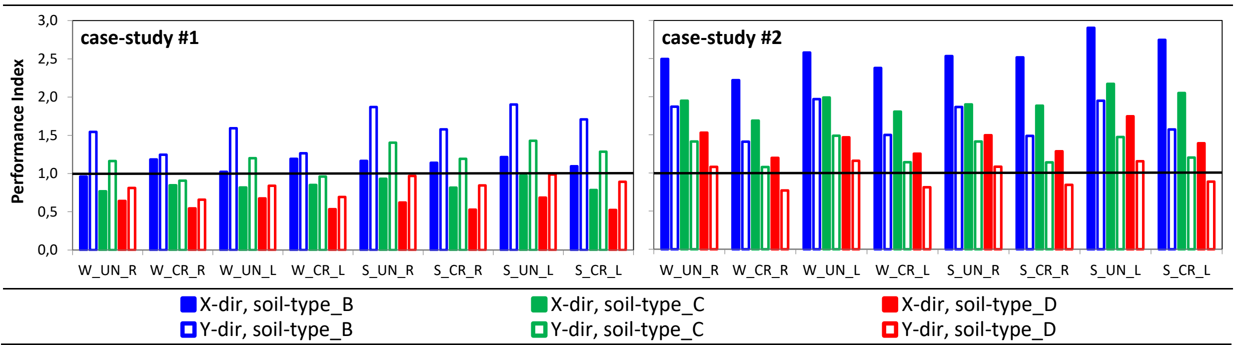


Figure 10. Performance Index of the two case-studies

The case-study #2 evidences higher values of *PI*, with the performance along the *X*-direction better than the one along the -direction. By assuming a soil-type B or C the building present an adequate safety level in all cases. When the D-soil is assumed, instead, the building performance along the *Y*-direction does not always comply to the safety requirements.

5 CONCLUSIVE REMARKS

In this paper the seismic assessment of masonry buildings representative of the XX century public housing interventions in Florence has been made. A classification of the buildings population belonging to the public housing activity has been proposed, based on aggregative criteria. One of the most common aggregation type consists of “block” buildings, rectangular in plan and with a number of storeys ranging between 2 and 4. Two case-studies have been selected to represent the block-type buildings family, having respectively 3 and 4 storey, and different orientation of floors and main masonry walls.

The seismic assessment of the case-studies has been made by performing a nonlinear static analysis, where the seismic input has been represented through the elastic spectra provided by the current Technical Code for a *Life Safety* limit state. Different soil classes, namely the B, C and D soil ones, have been assumed for setting the spectral demand, since the soil stratigraphy of the area presents a large variation due to the soil steadiness. Two different patterns, respectively proportional to the mass distribution and to the first vibrational mode, have been assumed for the pushover analysis. Furthermore, different hypotheses have been made regarding the strength and the stiffness of the masonry, and of the floor weight.

The analysis has evidenced a different performance of the two case-studies, with the case-study #2, having 3 storeys only, evidences a much better seismic performance. Furthermore, the assumptions made about the soil-type of the area plaid an important role in the seismic assessment of the buildings.

As regards the case-study #1, having the main walls along the *Y*-direction, the building is fully verified only for analysis in the *Y*-direction under the assumption of a B soil-type; at the opposite, when a D-soil is assumed for the area, the seismic performance is never satisfactory. For soil-class C, instead, the assessment depends on the assumptions made about the strength and the stiffness of the masonry. The variation in the floors weight, instead, does not affect the obtained results in any cases.

As regards the case-study #2, the safety requirement are satisfied in all cases in the hypothesis of soil-type B or C, while when the D-soil is assumed, the seismic performance of the building in the *X* direction.

The study has evidenced the possibility that the investigated buildings-type, i.e. the “block” masonry buildings, does not comply the current requirements concerning the seismic performance. Depending on *i)* the soil properties of the area, and *ii)* the specific features of the constructions, indeed, these buildings can be acceptable for their seismic performance. Their seismic assessment, therefore, should be investigated through more extensive analyses.

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