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Procedia Structural Integrity 11 (2018) 266–273

Structural Integrity Procedia

www.elsevier.com/locate/procedia

XIV International Conference on Building Pathology and Constructions Repair – CINPAR 2018

Public Housing in Florence: Seismic Assessment of Masonry Buildings

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Abstract

Florence is known all over the word for its historical and monumental buildings; however, most part of residential buildings have been made in the XX century, and they present all the critical issues proper of recent housing. Florence is one of the first Italian cities to have experienced the public housing. Its public housing population consists mostly of masonry buildings, and it does not comply the requirements provided by the current seismic legislation. This work is aimed at evaluating the seismic performance of a masonry building-type belonging to the public housing population of Florence. A typical public housing intervention, consisting of 18 masonry buildings, has been assumed as case-study. The seismic input has been described according to the Italian Code prescriptions, by considering the effective soil stratigraphy, and considering the effective mechanical properties of each layer. The seismic performance of the case-studies has been checked by performing a nonlinear static analysis. The results are expressed in terms of seismic performance, defined as the ratio between the seismic capacity and the corresponding demand. The obtained results evidenced the role of the building features on the seismic assessment of the buildings.

Peer-review under responsibility of the CINPAR 2018 organizers

Keywords: Public housing interventions; seismic assessment at urban scale; seismic performance of masonry buildings.

1. Introduction

Florence is mostly known for its outstanding historical architectures, which has deserved to become one of the World Heritage Site by UNESCO in 1982. Nevertheless, Florence is made of a large number of "recent" buildings,

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2452-3216
Peer-review under responsibility of the CINPAR 2018 organizers
10.1016/j.prostr.2018.11.035

since 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, and they would require a seismic assessment.

Within the residential buildings of Florence, the public housing interventions play an important role, since they have been constructed in Florence from the second half of the XIX century, when the town was – temporary - the Italian Capital (Bertelli *et al.* 2002; Pierini 2001). The buildings belonging to these public interventions present some common properties, such as to be more "trackable" and more "homogenized" than the private ones, since they have followed more severe building procedures and they have settled a standard in morphology, mechanical properties and – consequently – performance. In this number, the masonry buildings represent the majority of the public housing population. Such buildings population requires a careful assessment of the structural safety and the seismic assessment. To this purpose, a classification of the public housing population has been made, in order to distinguish building-types evidencing a different behavior under seismic excitation. In previous studies, (Metelli *et al.* 2017, Tanganelli *et al.* 2017) all the public interventions made in Florence in the time have been checked and classified as a function of their morphological and structural properties.

In this paper, a single intervention has been checked, belonging to the masonry public housing made in the beginning of the XX century. The assumed case-study consists of 18 masonry buildings, all of them having similar geometry and properties. The seismic performance of these buildings has been carefully checked, be representing numerically the seismic behavior of all them. They belong to the same building-type, according to the classification proposed in Metelli *et al.* (2017), but they differ from each other for their number of storeys, the in-plan walls distribution, the openings position and in the floors orientation. Eleven different structural models have been set, in order to closely represent the seismic behavior of all the buildings. The seismic capacity of the 11 models can be considered as representative of the behavior of many further similar interventions made in the Florence area. Therefore, the assessment of the seismic performance of the checked models provides important information regarding the safety condition of many masonry buildings of the town.

A nonlinear static analysis has been performed on the 11 models, in order to obtain the capacity curves of all of them. The capacity curves found for the models have been compared, in order to find the scatter due to the differences among the buildings in their seismic behavior. The capacity curves found for the models have been compared to the spectral demand of the seismic input, defined according to the Code (NTC 2008) prescriptions and to the mechanical properties found for the foundation soil. The seismic performance found for the 11 structural models have been shown and compared, in order to evidence the effects related to the buildings differences on their seismic performance.

2. The case studies

The assumed case-studies have been chosen after the classification made on the public housing population of Florence (Metelli *et al.* 2017); they belong to the same intervention, whose position within the Florence area is shown in Figure 1. Figure 2 shows the general plan of the intervention, with a 3D representation of all the masonry buildings. As can be noted by Figure 2, all the buildings have similar features; they have a rectangular plan, with a central stair which serves two apartments for each storey. In some cases, the buildings have been made through the combination of two (case-study #4) or three (case-study #7) single blocks.

All the buildings constituting the considered housing intervention have been made within the XX century. More precisely, as specified in Figure 2, the first 10 masonry buildings were made in the first half of the century, 9 further masonry buildings were made in 1951, whilst the last part of the intervention, consisting of RC buildings, was added successively. All the masonry buildings present similar technology and features. Anyway, the archive researches did not lead the same level of information for each building. For some case study a significant data of plan and technical specifications has been collected. For some other buildings, instead, not so many details are available, such as the case-studies #5, 9, 10 and 11, which have been modeled on the basis of the urban plans only, and the case-study #8, which has followed the plan presented in (Fantozzi *et al.* 2007) since no further material has been found regarding the masonry details. In all the buildings, 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. Two different types of masonry walls have been adopted, consisting respectively 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.

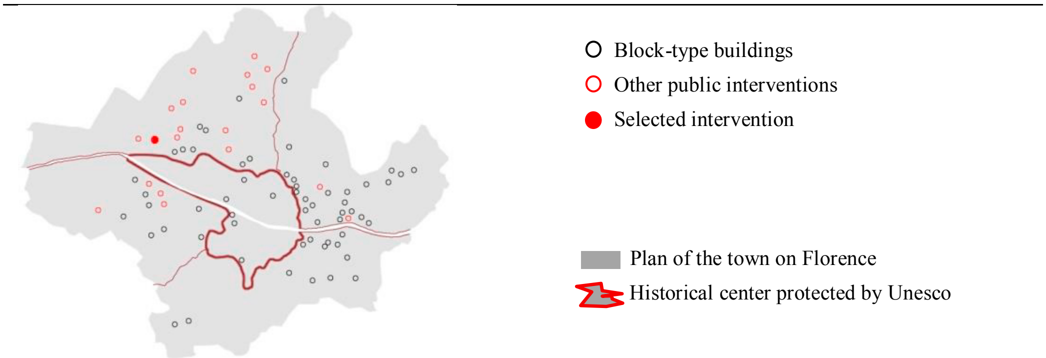


Fig. 1. Public housing interventions made in the town of Florence.

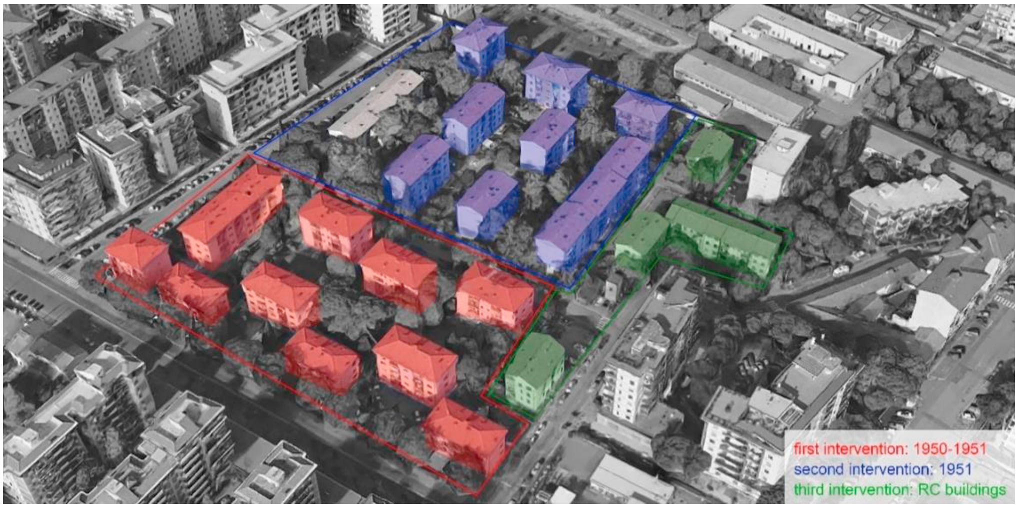


Fig. 2. The considered public housing intervention.

The plans of the 11 case-studies are listed in Figure 3. For sake of brevity, only the first storey plan of each building has been represented. As can be noted, all the buildings present similar features. Some of the models, however, present a longer length, since they consist of the combination of two (case-study #4) or three (case-study #7) basic blocks. The mechanical properties of the masonry have not been experimentally checked, and they have been assumed according to the information provided by the Technical Code (NTC 2008). Former studies (Metelli *et al.* 2017) evidenced that the variability in the mechanical properties of masonry can induce a large scatter on the seismic performance of buildings. Further variabilities checked in previous studies are related to the floors technology (Zaffi *et al.* 2017) and to the cracking status of the masonry (Tanganelli *et al*. 2018).

In this study, for sake of brevity, only the safest assumptions have been made, corresponding to a Confidence Factor equal to 1.35 (poor knowledge level); as a consequence, the masonry strength has been assumed through the most conservative value provided by the Code: the compressive strength, f_m, has been assumed equal to 260 N/cm^2 and 240 N/cm^2, respectively for the masonry-types A and B, whilst the shear stress, τ_0, has been assumed equal to 5.6 MPa and 6.0 N/cm^2 respectively. The Young modulus E_m has been quantified as the 50% of the initial one, i.e. by assuming a cracked behavior of the masonry. E_m, therefore, has been assumed equal to 1500 MPa and 1200 MPa, respectively, for the two masonry types. In Table 1 the main information of each model is listed. The quantities listed in Table 1 are, respectively, the total number of storeys, the direction of the floors disposition, the total length (x) and depth (y) of each block, together with their ratio (*x/y*) and the ratio between the resistant area of the masonry walls in the two main directions (A_x, A_y) and the area of the buildings plan (A).

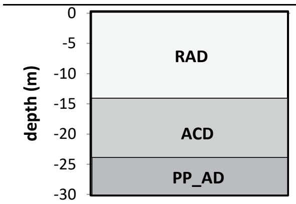
Fig. 3. Plans of the 11 case-studies.

Table 1. Main information regarding the considered case-studies.

case-study	Number of storeys	Floors disposition	*x* (m)	*y* (m)	*x/y*	$A_{res,x}/A$	$A_{res,y}/A$
case-study #1	3	transverse	20.50	10.1	2.03	8.5%	9.7%
case-study #2	4	longitudinal	21.80	10.5	2.08	5.9%	8.4%
case-study #3	4	crossed	21.50	11.7	1.84	8.0%	6.8%
case-study #4	5	crossed	38.70	11.7	3.31	7.8%	6.2%
case-study #5	5	crossed	15.25	11.8	1.29	4.9%	8.1%
case-study #6	5	crossed	21.50	10.6	2.03	9.0%	7.3%
case-study #7	5	crossed	63.60	10.6	6.00	8.9%	6.1%
case-study #8	5	crossed	23.00	9.8	2.35	7.5%	8.3%
case-study #9	5	transverse	16.85	10.5	1.60	8.1%	6.8%
case-study #10	5	crossed	19.60	13.1	1.50	5.4%	7.6%
case-study #11	6	crossed	19.15	11.9	1.61	6.1%	6.3%

3. The seismic input

The seismic input has been defined on the basis of the soil properties of the Florence area. In the past years. the soil of Florence has been checked through an extensive experimental investigation. which was based on the results of almost 2000 drillings. enhanced by 32 downhole proofs (Coli and Rubellini 2013). As a result of this survey. a grid has been obtained with the main information (Fundamental Period of the soil, bedrock depth and main stratigraphy) defined at each intersection. The soil profile of the considered area has been described by averaging the stratigraphies of the four corners of the grid of interest. In Figure 4 the stratigraphy represented the uppermost 30 meters has been shown. The upper layer of the stratigraphy consists of Recent Alluvional Deposits (RAD) of the Arno river and its tributaries. Experimental tests (Coli *et al.* 2015) showed that the shear velocity (v_s) of this soil, supposed to be cohesive, varies between 93 m/s and 639 m/s. The lower layer consists of Ancient Channel Deposits (ACD) of the Arno river and Plio-Pleiststocene palustrine; this soil, supposed to have a granular consistence, has a shear velocity ranging between 247 m/s and 965 m/s. The lowest layer, instead, consists of alluvial deposits (PP_AD) described by a granular behavior, and a shear velocity ranging between 320 m/s and 945 m/s. For each layer, an intermediate value has been assumed. The obtained $v_{s,30}$ has been compared to the soil classification provided by the current Italian Technical Code (NTC 2008), resulting to belong to the B-soil range (see Figure 4). This classification cannot be considered as exhaustive, since each layer has been described through an average value, not referring specifically to the area of the intervention; the possible variation in v_s of the layers, indeed, could provide alternative soil classification (see Tanganelli *et al.* 2018). Anyway, the B-soil is the most suitable soil-type of the considered area, and therefore it has been assumed as seismic input. The *Life Safety* limit state, i.e. a Return Period of 475 years, has been assumed, having a PGA equal to 0.13g. In Figure 5 the seismic spectra provided by NTC 2008 for six different Return Periods have been shown with reference for the considered area.



Assumed shear velocity (v_s) of the soil stratigraphy

soil type	v_s
RAD	341 m/s
ACD	663 m/s
PP_AD	734 m/s

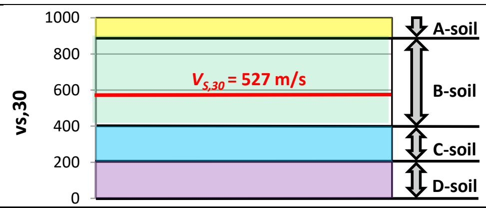


Fig. 4. Classification of the foundation soil according to NTC 2008.

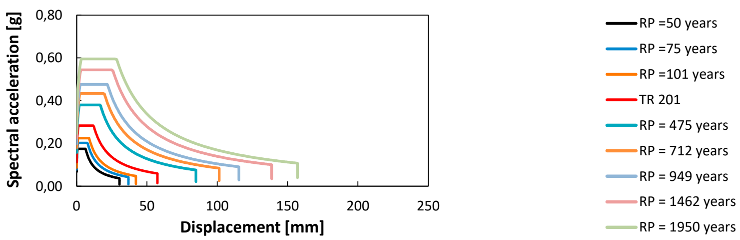


Fig. 5. Seismic spectra provided by NTC 2008 for the soil-type B.

4. The analysis

4.1. The seismic capacity

The capacity curves have been determined through the software 3Muri (STA DATA 2012), which is the commercial version of the Tremuri computer code developed by Lagomarsino *et al.* (2013). The masonry has been represented through plane panels with a nonlinear beam-behavior (Galasco *et al.* 2006). 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 influence of the floor deformability has been considered in the global response of the structural system. Two different distributions of horizontal forces have been considered, as required by the Code (NTC 2008), respectively proportional to the masses *(MP)* and to the 1st vibrational mode (*1P*). For each horizontal pattern, six analyses have been performed for each direction, to account for the accidental (equal to +/- 5%) eccentricity as required by the Code (NTC 2008). The capacity curves have been found by considering two different horizontal patterns, respectively proportional to the mass distribution and to the first vibrational mode.

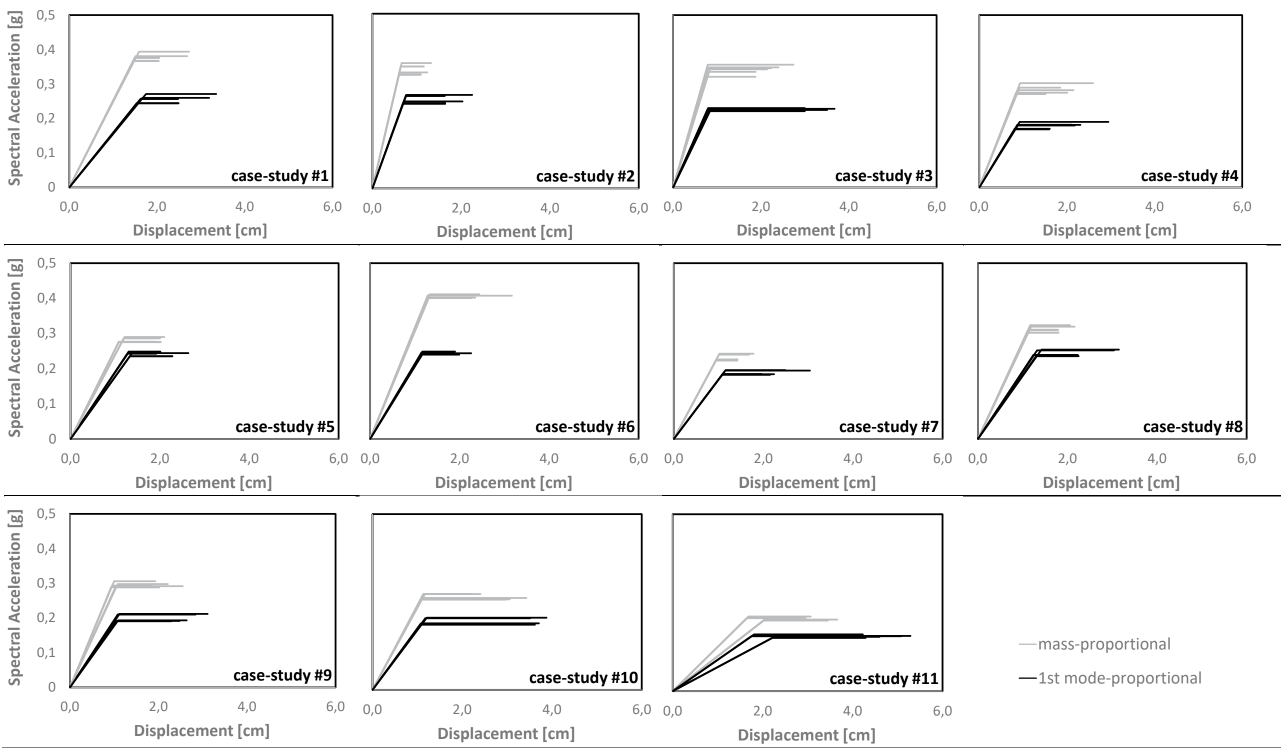


Fig. 6. Capacity curves of the 11 models.

The ultimate displacement of each curves refers to the lack of convergence of the model or a 15% drop of the maximum base-shear. The capacity curves have been expressed in terms of spectral acceleration and linearized according to the NTC 2008 provisions in order to be compared to the seismic spectral demand. In Figure 6 the capacity curves found for the 11 structural models have been shown. In Figure 7 the curves found for the 11 case-studies have been compared for the two considered directions and horizontal patterns, to better understand the effects in the capacity related to the differences among the buildings. As can be noted, the case-study #2 evidences a capacity much larger than the other ones along the *X*-direction, which is much stronger than the other, since the floors are oriented along the *Y*-direction only. The case-study #11, which is the tallest of the considered buildings, presents the lowest capacity, especially along the *Y*-direction.

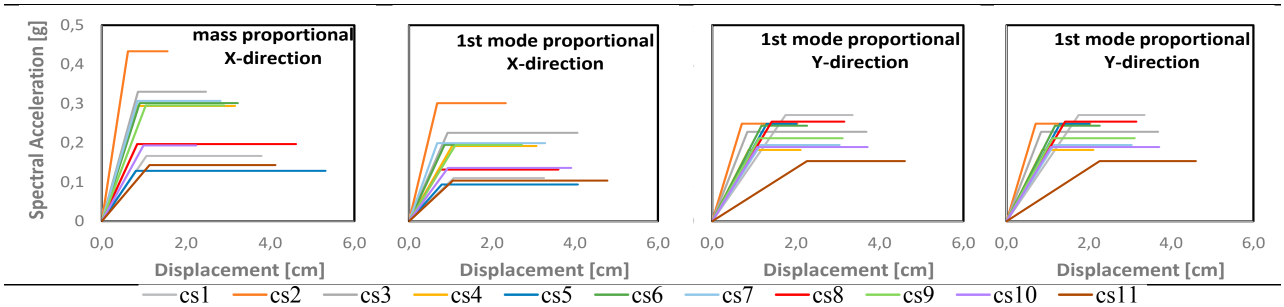


Fig. 7. Capacity curves of the 11 models.

4.2. The seismic performance

The capacity curves of the case-studies have been intersected to the spectral demand of the area for 9 different return periods. As a consequence, nine seismic responses have been found for each building at the varying of the seismic intensity. Each seismic response (D) have been compared to the corresponding capacity (C) of the buildings, defined according to NTC 2008 provisions and the assumption described in the Section 2. By comparing the seismic capacity of each model and the corresponding demand, the Performance Index (*PI*), expressing the accomplishment of the safety conditions required by NTC 2008, has been found. Figure 8 shows the Performance Index obtained for the 11 models. The Performance Index has been assumed as the minimum value between the ones found through the two considered horizontal patterns. As can be observed from the diagrams in Figure 8, in some of the buildings the performance index remains over the unity (i.e. the limit condition for the safety accomplishment) even for high seismic intensities, whilst some of them overcomes the limit even for low intensities. It should be noted that, in some cases, the limit conditions referred to the two considered (*LS*, *DL*) limit states are exceeded for the same seismic intensity. In Figure 9 the performance index found for the eleven buildings has been directly compared, to evidence the effects related to the differences among the buildings. In the Figure, the Performance Index related to the two limit states have been shown. The *PI* shown for each building is the minimum value provided from the analyses in the two directions.

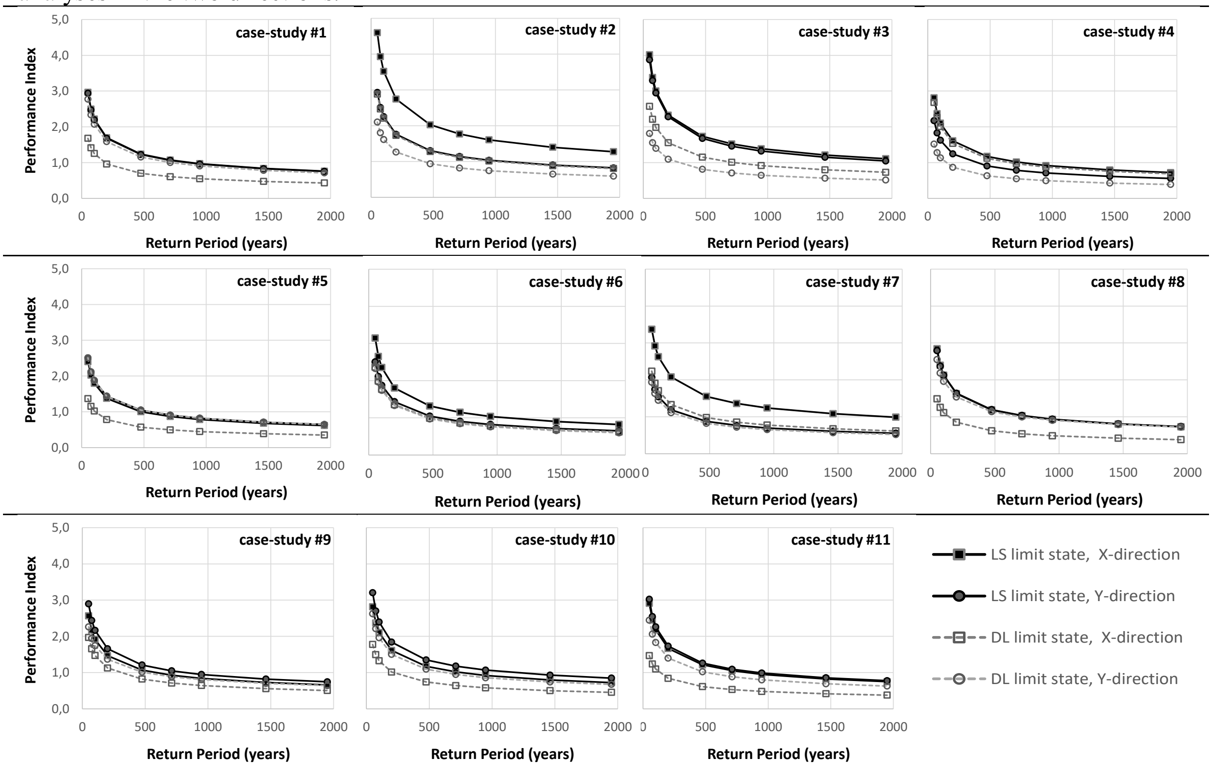


Fig. 8. Performance Index.

As regards the *Life Safety* limit state, the buildings exceed the safety limit provided by the NTC 2008 for Return Periods ranging between 350 years and 1200 years. Such range corresponds to a scatter in PGA between 0.115g and 0.17g, compatible to the seismic intensity of the area for the *LS* limit state. It can be observed that the case-studies #1, 2 and 3, which have 3 and 4 storeys only, respectively, present an high performance. The c.s. #3, which is the only "low" building to have a cross distribution of floors, presents the highest performance of the ample, remaining over the unity for the entire range of seismic intensity. As regards the *Damage Limitation* limit state, the performance index of the buildings exceeds the safety level provided by the Code for Return Periods ranging between 100 years and 450 buildings, corresponding to a PGA range between 0.08g and 1.23g, and therefore compatible with the considered limit state.

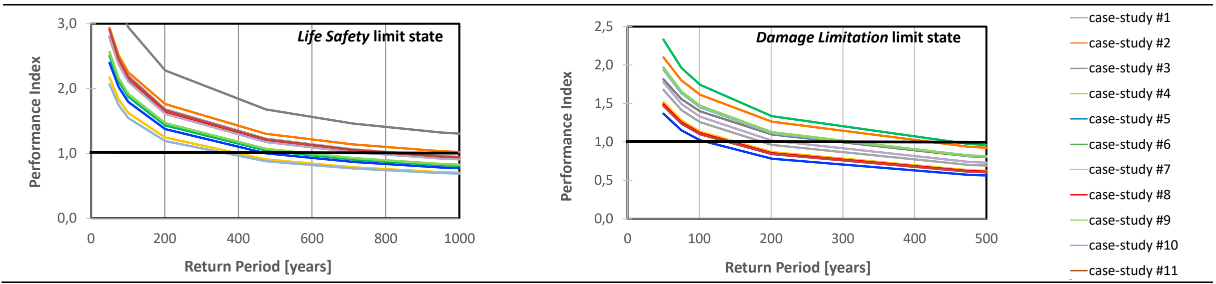


Fig. 9. Performance Index found for the 11 case-studies.

5. Conclusive remarks

In this work the seismic assessment of a public housing intervention, consisting of 18 masonry buildings, has been performed. The case-study intervention has been selected among the ones made in Florence in the XX century, and it is similar to many other interventions within the Florence area. The buildings constituting the considered intervention are very similar for their plan, material, technology and morphology, whilst they differ from each other for some plan details, number of storeys and mechanical percentage of masonry along the two main directions.

The seismic capacity of the buildings has been defined by performing a nonlinear static analysis along the two main directions, by assuming two different patterns of horizontal forces, whilst the seismic input has been assumed according to the NTC 2008 provisions, accounting for the seismicity of the area and its soil features.

The seismic response of each building, found by intersecting the spectral demand to the spectral capacity, has been compared to the limit values provided by the Code for two limit states, i.e. the *Life Safety* and the *Damage Limitation* ones. The seismic performance of the buildings has been expressed through the *Performance Index*, i.e. the minimum ratio between the capacity of their components and the corresponding seismic demand. By comparing the *PI* of the buildings it can be observed that they present important differences, for both the considered limit states. In order to achieve a synthetic representation of the building-type constituting the checked intervention, a more detailed classification of the buildings should be performed in terms of their main features, such as the number of storeys, the plan slenderness and the geometrical percentage of masonry along the two directions.

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