Seismic Vulnerability Assessment of a Building Aggregate in the Historical Centre of Florence

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Keywords: Aggregate Buildings, Masonry, Internal Court, Vulnerability Index Method, FEM Analysis

Abstract. Safeguarding the built heritage represents an urgent challenge for the culture and identity of each country. In Italy, past seismic events have highlighted the vulnerability of historic urban centres, as aggregates of historic masonry buildings. In this work, the seismic vulnerability of the historic centre of Florence, a UNESCO heritage site since 1982, will be investigated in the context of the Vulnerability Index Method, an empirical approach for the vulnerability assessment at the territorial level, proposed by Benedetti and Petrini in 1984, adopted by the Italian Group of Defense from Earthquake in 1993 and integrated by Formisano in 2011 with the key factors linked to the influence of the aggregate layout in the seismic behaviour. In particular, an urban aggregate composed of fourteen masonry *in-line* buildings (two palaces in the corner and twelve serial intercluded buildings) is considered as a case study. Buildings show a long narrow plan and an *internal court* and have undergone many transformations throughout history. Historical and typological analysis and material and constructive investigations were carried out to aid in understanding the mechanical behaviour of these buildings. These preliminary analyses allowed us to highlight the specific features and vulnerabilities of the aggregate, such as the presence of an *internal court*, which was the object of a specific study carried out supported by non-linear FEM investigations. In particular, this study was aimed at understanding how the GNDT form of the Seismic Vulnerability Level II can describe the vulnerability induced by the internal court in the seismic behaviour of the typical historical buildings in the city centre of Florence. First, the parameters of the GNDT form, influenced by the internal court, have been identified. Some considerations are reported by evaluating the results relating to these parameters, obtained for the application of the case study. Subsequently, some possible proposals for integrating the GNDT form were formulated to include the local vulnerability induced by the internal court in the structural behaviour of the typical historical buildings of the UNESCO city centre of Florence.

Introduction

Italy is characterized by a remarkable seismic risk, also due to the high vulnerability that characterizes historic urban centres. Studying the seismic behaviour of historic buildings is not simple as the behaviour of the individual building is conditioned by the aggregate condition itself and therefore by the interactions between adjacent buildings [1]. Furthermore, complexity is linked to the intrinsic characteristics of historical buildings made up of material and structural elements with non-linear behaviour influenced by a large variety of both geometric and mechanical factors [2; 3; 4].

In the regional seismic classification (GRT resolution 421/2014) [5], the city of Florence is located in zone 3, nevertheless, the territory falls into the high-risk class, as it contains one of the most important assets in the world declared a UNESCO heritage site in 1982.

The study of seismic vulnerability of historic buildings has been the subject of research since the early 1970s. In particular, various methodologies found in the literature [6; 7; 8] have been categorized into three different typologies based on data quality and nature. These typologies include:

- Analytical Method;
- Empirical Method;
- Hybrid Method.

In this study, the Seismic Vulnerability assessment methodology employed derives from the Seismic Vulnerability Index Method [6; 7; 8], an empirical approach for vulnerability assessment at the territorial level. This method was proposed by Benedetti and Petrini in 1984 [9], adopted and modified by the Italian Group of Defense from Earthquake in 1993 [10] and updated by the Tuscany Region in 2003 [11].

The methodology involves formulating a Seismic Vulnerability Index by specifying the vulnerability class relative to a series of parameters to which a weight is associated. All parameters are grouped in the *GNDT form of the Seismic Vulnerability Level II*, preceded by the GNDT form of the Seismic Vulnerability Level I, to frame the object of investigation in the urban context [10]. A fundamental contribution to the Seismic Vulnerability Index Method [6; 7; 8] is due to Formisano and co-authors [12], who in 2011 introduced five key factors linked to the influence of the aggregate layout in seismic behavior.

In this study, this empirical and expeditious approach is studied in relation to the considerations that emerged from the preliminary analysis conducted on the case study. In particular, the preliminary investigations carried out on the urban historic city centre of Florence, also supported by FEM analysis, have highlighted the seismic vulnerability induced by the *internal court*, generally recurring in the typical construction typology of the city.

The objective of this study was to evaluate the capacity of the *GNDT form* concerning the vulnerability due to the *internal court*. This vulnerability is not covered directly in the GNDT form of the Seismic Vulnerability, but the evaluation of some parameters is influenced by the presence of the *internal court*.

The study carried out focused precisely on how to integrate the *GNDT form* to include the construction specificity of the case study analysed, taking advantage of the fact that the form can be adapted to construction contexts with their characteristics. With this objective, similar methodologies, such as the American normative [13], which considers the *internal court*, have also been considered.

Vulnerability Index Method

The methodology involves defining a Seismic Vulnerability Index by specifying the vulnerability class relative to a series of parameters collected in the *GNDT form* of the Seismic Vulnerability Level II, provided by the National Group for Earthquake Defense [10].

Each parameter is associated with a weight, typically ranging from 0.25 to 1.5, depending on the parameter's influence on seismic vulnerability. For each parameter, a judgment must be expressed through four ascending vulnerability classes (A, B, C, D), each corresponding to a score ranging from 0 to 45. The sum of the products of the different vulnerability classes and the weight of each parameter allows for the derivation of the relative Seismic Vulnerability Index, Iv*. Normalizing this index within the range of 0 to 100, where 0 represents the absence of vulnerability and 100 represents maximum vulnerability, yields the Seismic Vulnerability Index.

Despite the limitations associated with the intrinsic subjectivity of the assessment, which relies on an expert judgment, this method does not define vulnerability based solely on the building's typology (as the macroseismic method does, which relies on the EMS-98 scale), allowing the vulnerability characteristics of the buildings under consideration to be determined in a specific way.

Furthermore, this methodology enables a large-scale qualitative assessment, facilitating an initial screening of constructed structures for the development of a management and action plan based on a priority scale for buildings considered to be at higher risk.

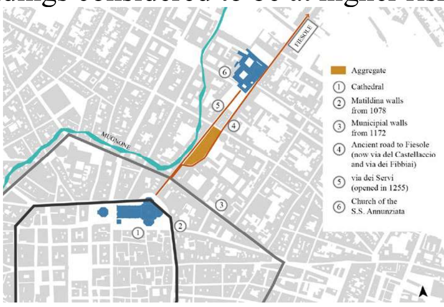

Figure 1. Urban plan with architectural emergencies and the identification of the aggregate.

Study Case

The aggregate object of the study is located near the Cathedral of Florence (Fig. 1) and is constituted of 14 buildings, two palaces occupy the head positions and 12 *serial intercluded buildings* are placed in between. The buildings were 13 in origin before a merge intervention of two adjacent buildings was carried out.

The aggregate shows an unusual shape, enclosed between two streets: *via dei Servi* and *via del Castellaccio*, which make the aggregate ideal for a line house typology, as a result of the transformations that occurred to the terraced house typology during the 19th Century [1; 14].

The serial buildings show a long narrow plan, accommodating the depth of the parcel, up to 40m. The depth-direction development of housing requires structurally bearing walls for both directions, orthogonal to the front streets as well as for the façades; the thickness of walls ranges around 45-50 cm.

The parcelling is not uniform: excluding the corner palaces, the first 4 buildings, from the left of Figure 2, show a width of 10 m, the 7 further houses show a width of 8,70 m, and the last two southmost parcels show a width of 9,57 m since they show a shorter depth compared to other buildings [15].

The presence of an *internal court* enables the sunlight to reach the internal rooms and is not in the centre of the plan. This position provides larger space for carrying out the main activities and gives increased brightness to the rooms facing the main street, via dei Servi.

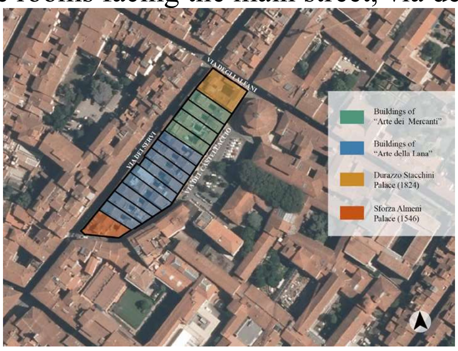

Figure 2. View of the roofs. Plan with the identification of different units: 9 belonging to Arte della Lana (blue) and 4 belonging to Arte dei Mercanti (green).

Preliminary investigation

Historical Framework. The aggregate falls into the core zone of the UNESCO area of Florence and was built just outside the walls of 1172-1175.

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Until the beginning of the 16th Century, the aggregate was occupied by an important *Tiratoio* (a building devoted to the production - tightening - of wood cloths), called *Tiratoio dell'Acquila*, who belonged to Noferi di Palla Strozzi, a rich banker of Florence. In 1417, he decided to pass the *Tiratoio* for 2/3 to the *Arte della Lana* and for 1/3 to the *Arte dei Mercanti*, two of the most prestigious and rich *Arti*, sort of professional societies and trade unions together [15].

Following the economic crisis that affected the textile industry, because of the deviation of the *Mugnone* river, the two Arti started a conversion of the area from production purposes to private housing, erecting 13 houses, 9 owned by the *Arte della Lana* and 4 owned of *Arte dei Mercanti* [16] (Fig. 2).

Typological Analysis. Three principal evolutionary phases of the aggregate can be identified: the first one (16th-17th Century) refers to the 16th Century unitary project of the *Arti* buildings; the second phase (18th-19th c.) refers to the first period of expansion, with the construction of elevations aligned and integrated with the masonry below; the third and last phase (from the 20th c.) refers to the smaller and back elevations. Despite the transformations that have taken place over the years, the original approach of the aggregate relating to the first evolutionary phase is still clear.

The typological reference model is the *court-terraced house*, which is single-family house. The study case shows variations of this typology connected to the attempt of increasing the available surface and implemented through saturation operations, i.e. development of the house in depth, with the annexation of further rooms up to the formation of a serial building with a *quintuple body* with double facing [17].

The typology is characterized by the presence of an *internal court* to guarantee adequate hygienic conditions of the rooms that cannot directly benefit from the lighting and ventilation from façades window.

Construction Technique. It was possible to define different types of masonry. In particular, the façade masonry, the masonry of longitudinal walls between buildings and the internal transverse walls, are all made of stone at least up to the second floor.

Superelevations (i.e. perimeter walls) at the third level are seldom implemented in stone masonry, while brickwork masonry is found as the unique technique on the third floor for internal partitions and renovation interventions.

There are also different types of floors: rooms on the ground floors and stairs are characterized by a vaulted roof; on the upper floors we can see wooden traditional floors, but also reinforced concrete slabs floors for the terraces referring to the recent elevations. For the roofs, the technique used is almost exclusively that of wood, with double or single-pitched roofs.

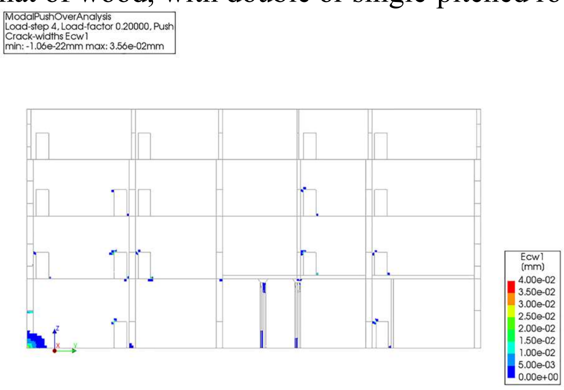

Figure 3. First configuration: the presence of the court is modelled thanks to the insertion of stone columns on the ground floor.

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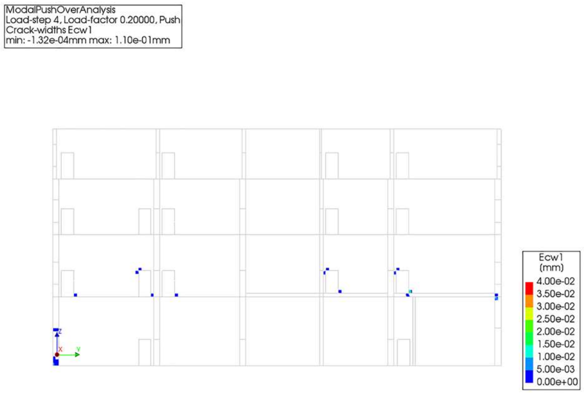

Figure 4. Second configuration: the presence of the court is modelled with walls also in the ground floor, in continuity with the walls in the higher levels.

Numerical analysis

The numerical investigation carried out on *finite element method* (FEM) models is aimed at highlighting the seismic vulnerability caused by the *internal court* of the building. The software used for model creation is *DIANA FEA*.

The modelling was carried out based on the information obtained during the preliminary investigation, particularly regarding wall thicknesses and mechanical properties associated with materials for each element. All models were fixed to the ground and fully constrained along the boundary walls in the transverse direction so that to resemble the containment effect of adjacent buildings. This constraint condition, although overestimating the rigidity provided by adjacent buildings, filtered the modes of vibration only in the longitudinal direction, i.e., the maximum length direction, orthogonal to the façades.

Initial modelling was created without considering the presence of floor slabs, an assumption made in favour of safety, given the deformability in the plane of the wooden floors. By neglecting the horizontal effect of slabs, a global response concentrated in the first vibration mode cannot be clearly identified.

On the other hand, it is possible to highlight the tendency of this kind of buildings to respond to seismic actions as a compound organism and through the activation of local damage mechanisms in the out-of-plane direction. Indeed, damage occurs due to vibration modes with lower frequencies, and involves small portions of the building and out-of-plane response of the masonry panels appears as predominant.

The second model of the building, which included the slabs, offers very different results. In particular, most of the participating mass is found in the first mode of vibration, so that a clear global behaviour can be identified. In this framework, expected damage is estimated at ground floor in the walls and at the corners of the internal opening due to a mainly in-plane response of the building. Also, the columns of the *internal court* undergo remarkable damage in either model.

Furthermore, two configurations with slabs were modelled: in both the slabs are perfectly clamped to the walls. In the first configuration, the presence of the court is modelled by the insertion of stone columns on the ground floor, resembling the current situation. In the second slab configuration, it is considered that the court has the same size, but it is defined by continuous walls rather than columns.

In the configuration with columns, at the same load step, the damage is more extensive, in particular, damage is also found at higher levels than in the first (Fig. 3; Fig. 4).

It is important to highlight that the assumption of perfectly anchored floor slabs to the walls does not describe the real condition of the building. Through the model, certain conditions are implicitly assumed in the best-case scenario. Consequently, the damage assessment emerging from the modelling is conservative but allows for defining the actual damage that will affect the real structure.

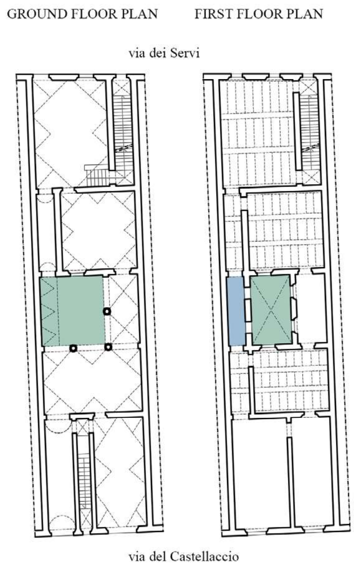

Figure 5. Reconstruction of the original configuration of the units: Ground floor (left) and First floor (right). Identification of the internal court (green) and of the additional cantilevered corridor on the first floor over the same internal court (blue).

Vulnerability associated to the presence of an inner court

Beginning with the *court-merchant house*, from which the *court-terraced house*, the reference typology for the buildings under study, derives, the expansion in the depth of the structure, along with subsequent added elements and adjunctions and the phenomenon of lot congestion, inevitably leads to the inclusion of an open space typically positioned centrally.

This space serves the purpose of illuminating and providing ventilation to the innermost areas. This element, referred to as a court, characterizes the majority of the historical architectural constructions in Florence, from smaller serial structures to grand noble palaces, where it reappear in larger dimensions.

In the buildings under examination, although the court belongs to the original layout, it inherently represents a significant point of discontinuity. Furthermore, the situation is exacerbated when considering the modifications the court has undergone during 19th Century practices, which altered its arrangement and functionality.

The presence of an *internal court* introduces irregularities at the floor level, which shows a hole that corresponds to the court and subsequent variations in the slab rigidity. Additional asymmetry and irregularity are observed in elevation. Specifically, on the ground floor, columns around the court must bear loads of the upper walls, highlighting structural discontinuity.

To this context, we must add the constructional weaknesses stemming from the evolution and modifications carried out over the years, beginning in the 19th Century. These include the construction of an additional cantilevered corridor on the first floor over the same *internal court* (Fig. 5).

These considerations make it impossible to associate the same stiffness to the cantilevered elements over the courtyard as to the other floor slabs. This extends the discontinuity and the vulnerability resulting not only from the *gap* created by the *internal court* but also to the portions of the floor slabs resulting from the gap itself, whose contribution to stiffness is negligible.

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Table 1. Parameters for the determination of Seismic Vulnerability Index for masonry buildings. Identification of the analyzed parameters (red).

PARAMETERS		CLASS				WEIGHT
		A	B	C	D	
1	Type and Organization of the structural system	0	5	20	45	1.5
2	Quality of the structural system	0	5	25	45	0.25
3	Conventional strength	0	5	25	45	1.5
4	Position of the building and type of foundation	0	5	25	45	0.75
5	Horizontal diaphragms	0	5	15	45	VAR
6	Plan irregularity	0	5	25	45	0.5
7	Height irregularity	0	5	25	45	VAR
8	Maximum distance between walls	0	5	25	45	0.25
9	Roof system	0	15	25	45	VAR
10	Non-structural elements	0	0	25	45	0.25
11	Physical condition	0	5	25	45	1

Vulnerability Index Method and internal court

In order to propose specific indicators capable of assessing vulnerabilities related to the court, this study presents a preliminary critical analysis conducted on those parameters from the *GNDT form* of the Seismic Vulnerability Level II [10], that are considered most useful to highlight the vulnerabilities connected to the presence of the *internal court* in the buildings (Table 1).

Parameter 3. Conventional Strength: Along with parameter *1. Type and Organization of the Resisting System*, this parameter holds the greatest weight within the *GNDT form*. The calculation considers the verification level, in general, the ground floor, assuming all floors are identical. However, this assumption does not hold when considering the presence of an *internal court* in the building. As demonstrated in the case study, the conventional strength calculated on the ground floor is different from the one calculated for the first floor, which is characterized by greater strength, even if only slightly, due to the presence of the walls corresponding to the columns below on the ground floor.

Parameter 5. Horizontal diaphragms. This parameter is based on two fundamental assumptions: the stiffness of horizontal diaphragms and the effectiveness of their connections with vertical elements. The evaluation is carried out on the general floor, and in cases of different types of horizontal elements, the condition defined by the worst-case scenario applies, provided it extends over a significant portion of the floor. The evaluation is done at a *global level* and is not suitable for interpreting the behaviour of the floor in the area of the court, in this specific case, where there is a change in geometry and certainly in stiffness. Furthermore, in this specific case, the nature of the connections changes, especially in the area of the cantilevered corridor (Fig. 5), supported by a beam.

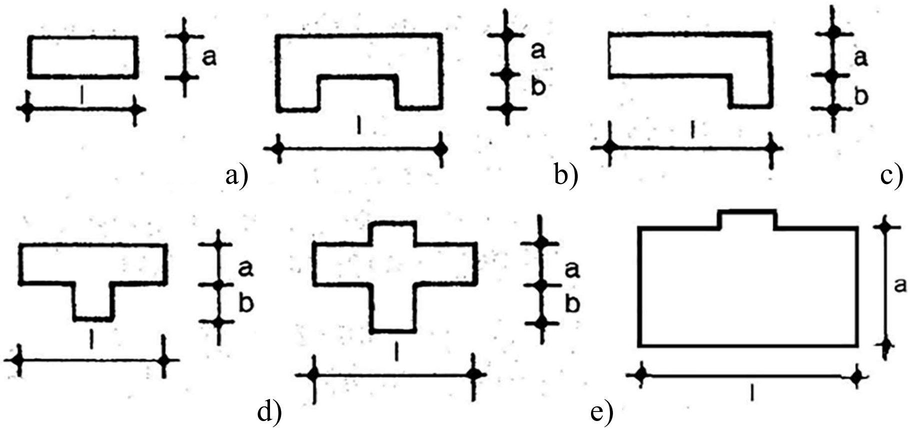

Figure 6. Plan irregularity. Schemes for calculating the plan irregularity β_2.

Parameter 6. Plan irregularity. This parameter depends on two factors: β_1 and β_2, representing the first one the ratio between the width and depth of the building and the second one the presence of plan irregularities. In addition to these, the *American seismic vulnerability form* [13] introduces an additional value to consider, which is the ratio between the area A_0 related to the court and the total area A_t, as well as the position of the court itself, central or lateral relative to the floor plan. As with the previous parameter, it is assumed that all floors are equal, and only the outer shape is considered. Moreover, articulations outside the outline <10% are considered inconsequential because, once again, the parameter focuses on global behaviour. The impact of changes in shape due to the presence of the court, when viewed globally over the total area, is not taken into account. The court, occupying a minimal area relative to the total area of the building, ends up being construed as a mere light well.

Parameter 7. Height irregularity. In masonry buildings, especially historical ones as in this case, the main cause of irregularities in elevation is precisely the presence of *portici* or *logge*. The parameter takes this into account, as well as changes in wall stiffness throughout the elevation of the building and any additions and reconstructions not contemporaneous with the original structure: once again, the behaviour is evaluated at a *global level*. For example, in this specific case, *logge* of modest dimensions, which affect less or at most 10% of the total area of the considered floor, lead to a downgrading concerning what could indeed be the level of vulnerability at the *local level*.

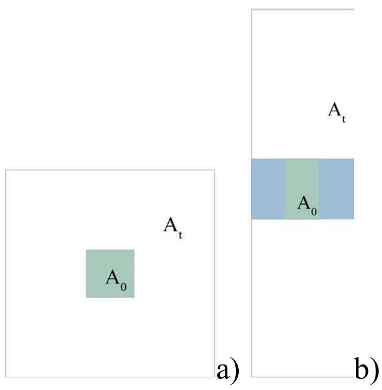

Figure 7. Plan irregularity. Distribution schemes to evaluate the transverse measure of the court in relation to the transverse measure of the plan. $A_0 = 20\ m^2$; $A_t = 314.64\ m^2$; $A_0/A_t = 6\%$. Identification of the cantilevered floor (blue).

Results

Several aspects emerge from the studied parameters that refer to global behaviour. The calculation of conventional strength performed on the model varies when comparing the ground floor to the first floor. When calculating the *coefficient C*, which represents the ratio between the ultimate shear T at the verification level and the weight P of the building portion above it, the value of the coefficient α for the ground floor is $\alpha = 0.15$, which is less than 0.4, where 0.4 is the reference value for seismic zones in the first category. For the first floor, $\alpha = 0.25$, also less than 0.4. Although this value also falls into Category D, it indicates a more adverse condition on the ground floor.

Regarding the plan irregularity, the ratio of the area of the *internal court* A_0 to the total area A_t is equal to 6%. For the calculation of the irregularity β_2 of the floor, which features an opening at the *internal court*, an approximate reference was made to the diagram in the manual (Fig. 6b), resulting in 9.8%, which is less than 10%, considered as the minimum value from the manual to indicate irregularity.

An assessment of the irregularity due to the presence of the *internal court* with respect to the floor slab could be made, not so much in relation to the A_0/A_t ratio, but to the ratio between the widths of the courtyard and the entire building. Taking into account the two models (Fig. 7), with $A_0 = 20$ m^2; $A_t = 314.64$ m^2, with an equal A_0/A_t ratio of 6%, considering the horizontal direction, model 7a) represents the better condition, in which the ratio between the width measurements is 0.2 (1:5), compared to case 7b) in which the width ratio is 0.3 (1:3), which becomes 1 (1:1) considering the further widening of the courtyard, considering that the slabs resting on beams do not offer stiffness comparable to the rest of the slabs, as in the case study.

In the height irregularity, we consider the portion affected by the *loggia*. This, concerning the total surface, is 5.7%, which is less than 10%, once again the minimum value to be taken into consideration.

Considering the issue at the *local level* and focusing the calculation on the individual cell, results naturally change. In the calculation of conventional resistance, while considering the ground floor, it remains in Category D with a value of $\alpha = 0.3$, less than 0.4. On the first floor, the value of $\alpha = 0.41$, which is greater than 0.4, falls into Category C. The same applies when considering the plan and height irregularities. In the first case, the A_0/A_t ratio is equal to 36%, and the β_2 coefficient is equal to 49.8%, which is greater than 10%, while the ratio of the area occupied by the *loggia* to the total surface is 34%, also greater than 10%.

Conclusion

Plan and elevation regularity are fundamental requirements for a better seismic response of a historical masonry building. Several factors can deviate a building from its regular configuration, and among these, the presence of an *internal court* has a fundamental role.

The structural response of the building is further worsened if the court is surrounded by a *loggia* which on the upper floors corresponds to closed spaces connecting adjacent rooms.

This construction system, in fact, implies an eccentric arrangement of resisting elements on the floor plan and a difference in the resisting area along the two principal directions. Furthermore, in elevation, a strong irregularity is represented by the different vertical structural elements around the court: columns on the ground floor and partition walls on the upper floors. The consequence of this irregularity also implies that 3 floors of partition walls weigh on a wooden beam supported by two stone columns.

This irregularity corresponds to the construction typology of the historic city centre of Florence. In this specific case, the court occupies the central core of the building which is also characterized by an extension in depth, much higher than the width.

This additional characteristic of the considered construction type results in the court's surface being small compared to the entire floor plan, but it practically occupies the central part of the floor plan's width, with negative effects on the seismic response concentrated mainly in the central part of the building. If the court is considered as a reduction in stiffness of the floor slab, the slabs adjacent to the court on the upper floors, resting on columns at the ground floor and not on continuous wall partitions, besides being the result of subsequent interventions compared to the original layout, cannot be assumed to have the same stiffness as the others.

In contrast to American normative that refer to *Diaphragm Discontinuity* [13], describing irregularities related to open portions of the floor structure with consequent changes in rigidity, the GNDT Seismic Vulnerability Index Method Level II [10], does not explicitly consider the case of the presence of a court, although many parameters consider irregularities induced by the presence of an internal court.

Considering the specific analysed case, the small size of the court compared to the total area means that even the criterion of [13], referring to the ratio of the area of the *internal court* A_0 to the total area A_t, does not capture its vulnerability.

In light of these considerations, it may be useful, in terms of results, to assess vulnerability by considering the transverse direction, the main weak axis if considering seismic action in the same direction. A possible modification to the parameter could be to specifically consider the ratio between width dimensions.

As revealed from the examination of parameters in the *GNDT form* (Table 1), considered most suitable and closely related to the interpretation of the vulnerabilities connected to the presence of the *internal court* in the buildings, the primary problem in the seismic vulnerability assessment referring to an *internal court* in the building, is related to the seismic vulnerability assessment of the building at a *global level*.

To investigate the onset of damage in correspondence to the internal courtyard of the building, a *localized* study would be necessary, considering the parameters of the *GNDT form* relating to the single cell where the presence of the court has the greatest impact.

In this study, a correlation between assessments of seismic vulnerability at the *global level* and at the individual element level is proposed.

References

[1] G. Caniggia, G. L. Maffei. Composizione architettonica e tipologia edilizia 1/2. Il progetto nell’edilizia di base. Marsilio editori, Venezia, 1979.

[2] N. Chieffo, (et al.). A vulnerability index-based approach for the historical centre of the city of Latronico (Potenza, Southern Italy), in Engineering Failure Analysis, n. 136. 2022. https://doi.org/10.1016/j.engfailanal.2022.106207

[3] A. De Maria, C. Donà. (a cura di). Manuale delle murature storiche. Analisi e valutazione del comportamento strutturale. Volume I. DEI Editore, Roma, 2011.

[4] A. Giuffrè. Letture sulla meccanica delle murature storiche. Edizioni Kappa, Roma, 1991.

[5] G. A. Centauro, C. Francini (a cura di). Progetto HECO (Heritage Colors). Metodologie Analisi Sintesi Apparati Valutazione d’impatto sul sito UNESCO Centro Storico di Firenze. DiDaPress, Firenze, 2017.

[6] A. Preciado (et al.). Earthquake risk assessment methods of unreinforced masonry structures: Hazard and vulnerability, in Earthquakes and Structures. 2015. https://doi.org/10.12989/eas.2015.9.4.719

[7] R. Maio. Seismic vulnerability assessment of old building aggregates. Master’s thesis. University of Aveiro. 2013.

[8] G. M. Calvi (et al.). Development of Seismic Vulnerability Assessment Methodologies Over the past 30 years, in ISET Journal of Earthquake Technology, N. 472, Vol. 43. 2006.

[9] D. Benedetti, V. Petrini. Sulla vulnerabilità sismica di edifici in muratura: un metodo di valutazione. A method for evaluating the seismic vulnerability of masonry buildings, in L'industria delle Costruzioni, n.149. 1984.

[10] GNDT. Rischio sismico di edifici pubblici- Parte I. Proceeding of CNR- GNDT, Roma, 1993.

[11] M. Ferrini (et al.). Rilevamento della vulnerabilità sismica degli edifici in muratura. Manuale per la compilazione della Scheda GNDT/CNR di II Livello (versione modificata). Regione Toscana, 2003.

[12] A. Formisano (et al.). Un Metodo per la Valutazione su larga scala della Vulnerabilità Sismica degli aggregati storici. Anidis, Bari, 2011.

[13] National Institute of Building Sciences. NEHRP Recommended Seismic Provisions for new Buildings and other structures. FEMA, 2020.

[14] G. L. Maffei. La casa fiorentina nella storia della città dalle origini all'Ottocento. Marsilio editori, Venezia, 1990.

[15] G. Cataldi. Palazzetti a schiera in via dei Servi a Firenze, in Studi e documenti di architettura, n.14, Firenze, 1987.

[16] C. Tomasini Pietramellara. Tipologie edilizie, in Firenze, studi e ricerche sul centro antico. Nistri-Lischi, Pisa, 1974.

[17] C. Chiappi, G. Villa. Tipo/progetto/composizione architettonica. Alinea editore, Firenze, 1980.

[18] MIT. Circolare 21/01/2019. Istruzioni per l'applicazione dell'aggiornamento delle Norme Tecniche per le costruzioni, di cui al decreto ministeriale 17/01/2018. 2019.