Seismic Pounding Analysis of Palazzotto Borbonico "Vecchio" and "Nuovo" in Naples

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Abstract. Recent investigations of past earthquake damages in Italy highlighted the importance of assessing the seismic pounding between adjacent buildings. The problem of the safety verification of adjacent buildings is becoming extremely relevant as pounding effects may be highly destructive. The seismic vulnerability connected to this phenomenon is usually increased when the adjacent buildings present different construction materials and were realized before building codes prescribed a minimum size for separation joints in seismic areas. The case study which is here discussed refers to the city of Naples, Italy, a seismically active site, and to a couple of buildings which were built in different centuries and present different structural materials, i.e., masonry and timber for the historical one, called "Palazzotto Borbonico Vecchio", and reinforced concrete for the modern one, "Palazzotto Nuovo". Such buildings constitute two independent dynamic units, as it is clearly shown by the separation joint which was spontaneously generated along the bearing masonry wall that they share. The structures have been analyzed by means of three Finite Element models, both individually and joined; the global F.E.M. allows to consider the non-linear effect due to the buildings collision. Several linear and non-linear dynamic analyses have been performed in order to investigate the correct separation distance required for avoiding the pounding phenomenon, and the effects of pounding, varying parametrically the gap distance and the stiffness of the link elements.

Keywords: Pounding · Structural analysis · Historical building
Impact model · Seismic analysis

1 Introduction

Pounding between adjacent buildings occurs when, due to an external dynamic excitation, usually an earthquake, the structural systems behave like independent dynamic units and separation joints are not adequate to receive the relative displacements that arise. This research presents an emblematic case study that relates to two adjacent structures with totally different dynamic characteristics as they were built in distinct

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centuries and materials. In this case, the historical building was realized in masonry while the modern one in reinforced concrete.

The performed seismic pounding analysis shows that the unequal dynamic behavior of the two structures generated, over time, a spontaneous separation joint located along the masonry bearing wall that represents the interface between the two adjacent buildings.

In this study the pounding phenomenon between Palazzotto Borbonico Vecchio and Nuovo has been investigated through a non-linear analysis of the collision phenomenon, in order to evaluate both the minimum gap required to prevent the phenomenon to occur and the variation of impact loads at interaction points as well.

2 Architectural Survey and Material Testing

2.1 Historical Analysis and Architectural Representation

The complex, called "Palazzo Borbonico", is an historical building located in the Capodimonte Park in Naples, Italy [1]. The building is composed of two adjacent structures: the ancient one, called "Palazzotto Vecchio", is made of bearing masonry walls while the recent one, named "Palazzotto Nuovo", is constituted by a reinforced concrete tridimensional frame. The comparison of some historical maps related to the evolution of the city highlighted that, since the beginning of 19th century, the building complex was already present in that urban area and was entirely made of masonry. The new reinforced concrete frame structure was realized as a consequence of the damages produced by World War 2 bombing. The architectural survey pointed out the regularity of the ancient structure, that presents a rectangular plan (dimensions 24 $\times$ 15 m) made of bearing masonry walls with relevant thickness and good construction quality, oriented in two main orthogonal directions in plan. The floor system, partially rebuilt over the centuries, presents a wide heterogeneity at the different floor levels: timber structures, reinforced concrete and mixed. In the new structure, foundations consist of isolated plinths while the slabs are in hollow-core concrete, with a thickness of 30 cm.

2.2 Mechanical Properties for Masonry and Reinforced Concrete

According to the Italian Building Code, the experimental campaign to assess the mechanical properties of materials represents the preliminary condition to an adequate level of knowledge of an existing structure and, therefore, to a good confidence factor. An experimental campaign of non-destructive material testing has been performed; it included thermography, endoscopy, single and double flat jacks [1]. The thermographic analysis played an essential role to estimate the location of structural elements that were not visible (as embedded in non structural brick infill) and understand the brick texture of the masonry walls.

The sonic analysis, in combination with the single and double flat jacks, highlighted the masonry stratigraphy and its mechanical characterization. All the performed tests showed that the masonry walls of Palazzotto Vecchio are in tuff; according to the

Table 1. Masonry and Reinforced concrete mechanical properties.

Masonry				
E [Mpa]	G [Mpa]	W [KN/mc]	τ_0 [Mpa]	f_m [Mpa]
1620	520	16	4,68	255
Reinforced cocnrerete C 25/30				
E [Mpa]	G [Mpa]	W [KN/mc]	f_{cd} [Mpa]	f_{ctd} [Mpa]
31500	13100	25	14,17	1,20

statistical processing of the results, data in Table 1 represent the mechanical parameters used for the numerical analysis.

3 Numerical Models and Analysis

3.1 Definition of the Seismic Action

In order to analyze the seismic response of the Palazzotto Borbonico complex, the elastic response spectrum prescribed by the Italian Building Code for the specific site with a return period of 475 years has been initially considered. Subsequently, on the basis of both experimental and numerical analyses, the local seismic response has been characterized, related to the geological, stratigraphic and topographic site conditions, see Fig. 1. In addition to the linear response spectrum analysis, dynamic non-linear analyses have also been performed, through the use of acceleration time histories generated from the local response spectrum [2].

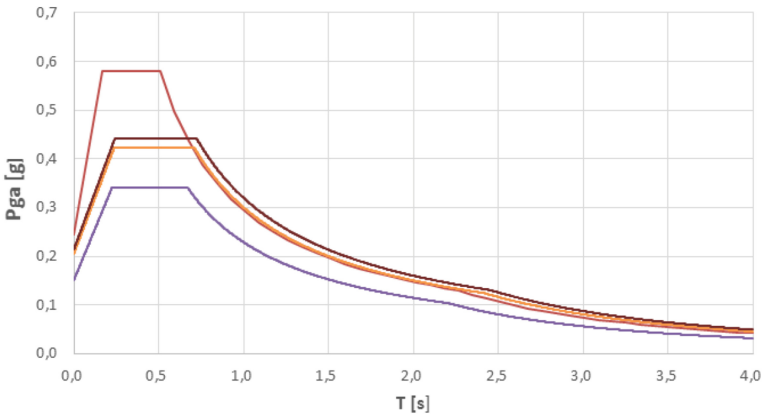

Fig. 1. Different response spectra considered in the numerical analyses: code spectrum (the highest, in red, for a return period of 475 years) and local spectra (corresponding to return periods of 712, 475, and 201 years, respectively).

3.2 Numerical Model of the Structural System

The analysis of the dynamic interaction between the two adjacent buildings required the development of a global numerical model including both the masonry structure and the reinforced concrete one (Fig. 2). The Finite Element software used is SAP2000, through which the simulation of the impact action between different structures is possible. The seismic response of Palazzotto Vecchio has been previously analyzed in detail for different accuracy levels and through a series of simulations, using the "Tremuri" software [3], specifically developed for the analysis of masonry systems. This numerical model has been used to calibrate the equivalent Finite Element model in the SAP2000 software, which is more oriented to reproduce the actual geometry but not properly designed to analyze masonry structures. The calibration of the SAP2000 model has been based on the modal analysis and, specifically, on the periods of the natural modes.

3.3 Dynamic Analysis

To the purpose of analyzing the pounding conditions of the two buildings, maximum horizontal displacements have been estimated. Both response spectrum modal analyses and time history analyses have been used; load combinations include, in addition to vertical loads, the earthquake load in each direction plus 30% of the load in the orthogonal direction. Typical results are shown in Fig. 2, in terms of maximum displacement values at the story levels for each building. The minimum distance to avoid pounding can be computed considering displacements normal to the interface wall (see Fig. 6), which are given by the following combination:

$$u_x^{'} = u_x \cdot \cos(13^\circ) + u_y \cdot \cos(90^\circ - 13^\circ) \tag{1}$$

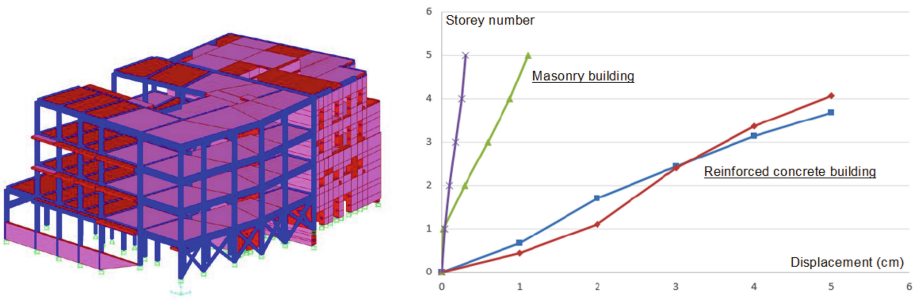

Fig. 2. The global structural model (left) and displacements at storey levels at the interface (right).

The minimum length for the separation joint is given by the sum of the absolute values of maximum displacements computed for each building:

$$d_g = |u'_{x,old}| + |u'_{x,new}| \tag{2}$$

Results are given in Table 2 for each load combination. The computed values have to be compared to the minimum size prescribed by the Italian Building Code for the separation joint, to be evaluated as:

$$d_g \geq a_g \cdot S/0.5 \cdot g = 10\,\text{cm} \tag{3}$$

where $a_g \cdot S$ is the peak ground acceleration amplified by the soil factor and g is the gravity acceleration. From the previous analysis, the following conclusions can be drawn:

- the requirement for the separation joint, as specified in the Italian Code, is on the safe side, but not unrealistic;
- in terms of computed displacements, the response spectrum provided by the analysis of the local seismic response is less severe than the reference response spectrum given, for the specific site, by the national map of seismic hazard.

Table 2. Minimum values for the separation joint (d_g), according to different analyses

Type of analysis	d_g [cm]
Response spectrum – National code	8
Response spectrum – Local (Return period = 201 years)	5
Response spectrum – Local (Return period = 475 years)	6
Response spectrum – Local (Return period = 712 years)	6
Time history analysis	7

4 Non-linear Dynamic Analyses

4.1 The "Fast Non-linear Analysis" Method

After analyzing the seismic response of the two buildings as independent dynamic systems, the seismic interaction has been investigated [4, 5], in terms of pounding effects and consequent collision forces. To this purpose, special elements characterized by a non-linear behaviour have been inserted in the finite element model at some locations, where interaction is assumed to take place.

The solution to the non-linear problem is obtained through the method proposed by Wilson [6], called "Fast Non-Linear Analysis" (FNA), suitable for the analysis of problems characterized by a limited number of elements exhibiting a non-linear behaviour. In such cases, using this method is convenient, as the computational time is strongly reduced and results do not loose in accuracy.

The FNA method is based on the classical equations expressing the dynamic equilibrium of an N degree of freedom system, to which a special vector is added including all the nodal forces corresponding to the non-linear elements. The following form, therefore, is assumed by the equations of motion:

$$M \cdot \ddot{u}(t) + C \cdot \dot{u}(t) + K \cdot u(t) + R_{NL}(t) = f(t) \tag{4}$$

where M, C and K denote the mass, damping and stiffness matrixes, $u(t)$ the vector of the displacement functions and $R_{NL}(t)$ a force vector, including the nodal loads coming from the non-linear elements. In case equilibrium necessarily requires the presence of these last elements in the finite element mesh, they can be added to the system specifying arbitrary values for stiffness (K_e); at the same time, elastic forces $F_e(t) = K_e \cdot u(t)$ are added. The dynamic equilibrium equations, therefore, can be written as:

$$M \cdot \ddot{u}(t) + C \cdot \dot{u}(t) + (K + K_e) \cdot u(t) = f(t) - R_{NL}(t) + F_e(t) \tag{5}$$

Assuming: $\tilde{K} = K + K_e$ and $\tilde{f}(t) = f(t) - R_{NL}(t) + F_e$
the final formulation is:

$$M \cdot \ddot{u}(t) + C \cdot \dot{u}(t) + \widetilde{K} \cdot u(t) = \widetilde{f}(t) \tag{6}$$

In this way, dynamic equilibrium is satisfied. If a suitable value is assumed for the elastic stiffness of non-linear elements, convergence is fast and the method comes out to be computationally efficient.

4.2 Main Results

Eight non-linear elements have been included in the model, two for each floor; they are located, at the interface between the two buildings, where facing nodes exhibit maximum relative displacements, see Fig. 3.

A parametric study has been performed, analyzing the effects of both the parameters characterizing the contact elements: the gap distance and the stiffness developed at collision. For the first parameter, the values of 0.5, 1, 2, 3 cm have been considered. As to the stiffness value, the indications given by Maison and Ksai [7, 8] have been followed; the suggested range of values for r.c. structures subject to pounding is 15,000–50,000 kips/inch. In consideration of the higher stiffness developed by the masonry structure, the suggested range has been amplified; the following specific values have been adopted: 2500, 5000, 8750, 10,000 MN/m. Typical results are shown for a specific case in Fig. 4; subsequently, a global view of the pounding effect is given.

From the point of view of local effects, it can be observed that impact loads generated by seismic pounding may be extremely high [9]. This result, indeed, is in line with those coming from all the mechanical models discussed in the literature. In terms of global effects, Fig. 5 shows the variation in time of base shear and global overturning moment, comparing the situations where pounding is present or not. The sudden significant increase of base actions at collision time can easily be observed.

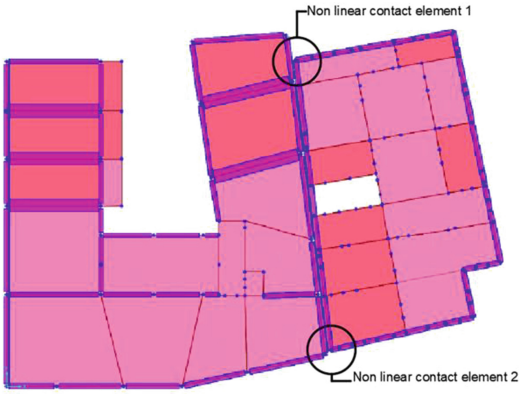

Fig. 3. Location of the non-linear elements at the interface between the two buildings.

With reference to all the analyzed cases, the diagram shown in Fig. 6 has been obtained. The diagram shows the number of collisions as a function of both stiffness and gap distance. It can be seen that the phenomenon undergoes high amplification as the gap distance decreases and, at the same time, stiffness values become low. As to the element stiffness, a critical minimum value in the range 5,000–10,000 MN/m can be recognized.

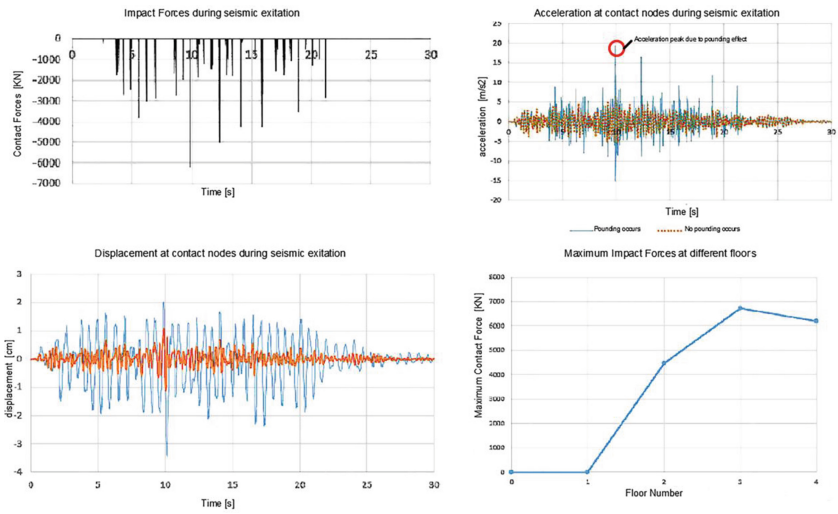

Fig. 4. Non-linear dynamic analyses: results for a specific case.

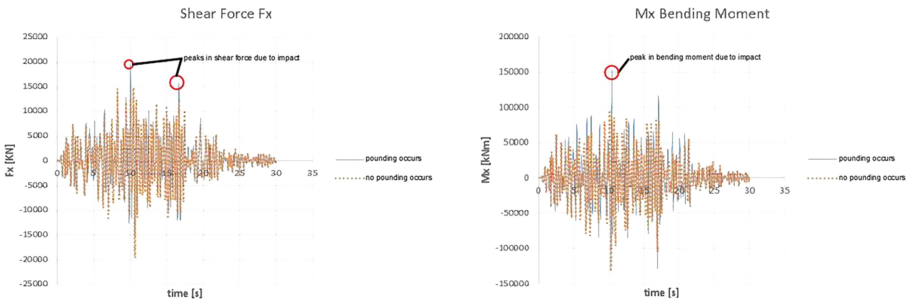

Fig. 5. Base shear and overturning moment in the presence of pounding.

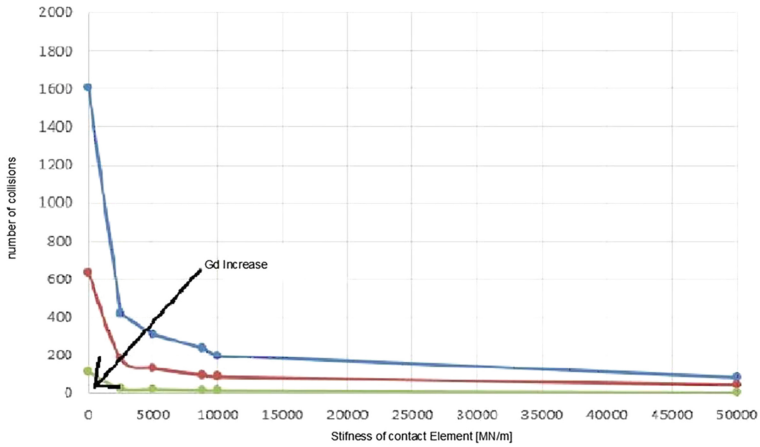

Fig. 6. Number of collisions during the seismic event as a function of stiffness and gap distance (G_d).

5 Conclusions

The evaluation of seismic safety for existing buildings should also consider pounding as a relevant phenomenon, whenever adjacent buildings behave like independent dynamic systems and separation joints are not present. Such a situation is very common, as separation joints normally were not considered before seismic design prescriptions were introduced.

Through the analysis of the selected case study, it has been shown that the code requirements in terms of minimum gap between buildings, although conservative, are realistic; at the same time, non-linear analyses of the collision phenomenon have led to the evaluation of significant impact loads at interaction points.

The complexity of the pounding phenomenon, which is non-linear by nature, calls for the necessity of more extended studies, considering a variety of physical impact models at different sophistication levels. From the analyses here performed, as a first result, it appears that reducing the gap distance and increasing, at the same time,

stiffness at the interaction points reduces the number of collisions. In the limit case, providing full connection and collaboration between the structural units might lead to a more favourable seismic response.

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