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# SEISMIC ANALYSIS AND RETROFIT OF A MID-RISE BUILDING

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**Abstract.** *The structural analysis of an about 30 m tall building is presented in this paper. It was constructed in 70s, so its use pointed out the issue of the seismic reliability of existing buildings, especially if considered of strategic importance for civil protection. The structure is composed by two parts, the lower one in reinforced concrete, which contains a theatre room, and an upper in steel. The study allowed defining the seismic vulnerability. A refined finite element model was assembled and updated on the basis of comprehensive experimental analyses on materials and structure and also of experimental modal analysis, highlighting the dynamic effect due to the irregularity of the building and the influence of the adjacent building, the seismic joint is missing. The results pointed out a quite high vulnerability of the building to seismic actions and the inadequacy of the pile foundation to support horizontal seismic loads. A retrofit intervention, based on the insertion of a seismic isolation system and the realization of a seismic joint with the adjacent building, is proposed. It allows reducing the horizontal forces in the piles and the torsional movements of the structure.*

## 1 INTRODUCTION

The Italian territory seismic classification started after the 1908 Messina and Reggio Calabria earthquake (M=7.1) and was updated only after new seismic events up to recently. So, most of the Italian structures were designed without accounting for the seismic actions or, in some cases, with design earthquake of intensity lower than that required in the present code. This could be very dangerous for strategic structures, such as buildings with civil protection purposes, bridges and railways critical to the transport networks, plants at risk of a major accident, etc., and of relevant structures that can be very crowded, such as schools and hospitals. Also the recent Italian earthquakes pointed out the high vulnerability of a large number of structures and the strategic and relevant ones are no exceptions [1], making the overall improvement of the built very hard. Only recently, the reliability evaluation and the design of improving interventions of existing strategic and relevant structures have been considered compulsory in the framework of a prevention policy.

The improvement process of a singular structure consists essentially in two steps. The first one consists in the evaluation of its structural health status. This can be performed on the basis of an appropriate experimental analysis, which includes material and structural tests, and of an accurate numerical analysis with a suitable model, which accounts for the actual characteristics of the structure obtained experimentally. In the second step a proper intervention should be designed on the basis of the experimental and numerical results of the first step.

In this paper the analysis carried out on an existing building is shown. The original design being available, it was quite easy to perform a suitable finite element model. Then the available experimental results were analyzed and a more detailed experimental campaign was organized, which included test on the soil and foundations, on steel and concrete elements and ambient vibration analysis to extract the dynamic properties of the building. The updated model allowed analyzing the actual capacity of the structure, highlighting the irregular dynamic behavior of the building under seismic load and the inability of the foundation to resist horizontal actions, and designing the most adequate retrofit intervention, which consists in the insertion of a base isolation system.

## 2 THE BUILDING

The building object of this study, built in the 70s in an area not considered as seismic zone at the time of construction, has eight stories above the ground (floor 0, floors 1 to 7, respectively) and one underground story of shorter height (floor -1). From the architectural point of view the building is composed by two different parts in elevation (Figure 1). The lower part has in-plan squared shape and includes the underground level and three levels above the ground (floors 0 to 2 – Figure 2 left). The upper part, where are located offices, has a T shape covering only a portion of the lower part (floors 3 to 7 – Figure 2 right). The lower part contains an auditorium and a boardroom, therefore the structure should be considered as building of particular relevance. The four steel columns, which are around the auditorium (Figure 2 left), and the corresponding columns at the north and south sides (two for each side) support a spatial Vierendeel system, whose height occupies the entire floor 3. Most of the columns of the seventh level are not in correspondence of other columns but are supported by the lower beams. The stability against horizontal actions and second order effects is guaranteed by concrete shear walls, placed eccentrically with reference to the gravity center at the south side in the WE direction and around the staircase. The presence of stiff end walls leads to the needs of considering in the analysis the floor diaphragm flexibility [2][3].

The structure is adjacent at the south side to another building of lower height. Given that no seismic joint was made at the construction age, the two structures are exposed to the risk of pounding in case of earthquake [4].

The load-carrying structure is mainly constituted by steel members, joined by welding. It was designed according to the Italian code existing at the construction time and therefore without accounting for seismic actions and without adopting usual details for structures in seismic area. In the lower part also concrete members are present. The foundations consist in concrete plinths with piles of about 40 m length, with reinforcing bars only in the first 16 m. The plinths are connected by means of concrete beams.

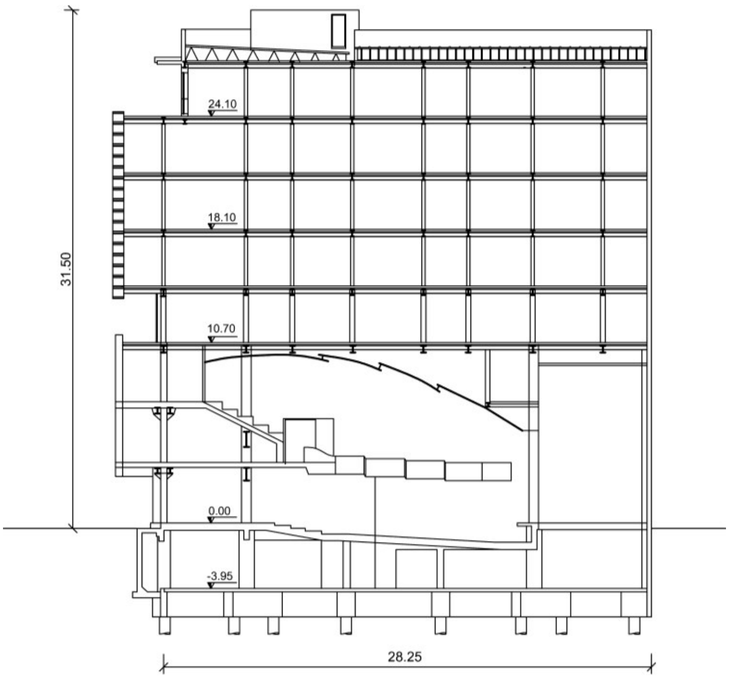



Figure 1 : Vertical section of the building (North side on the left)

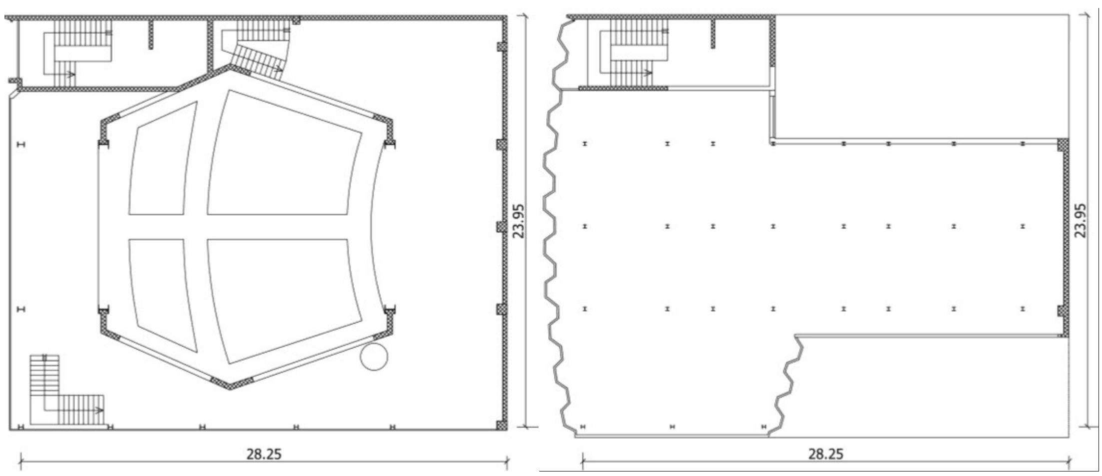



Figure 2 : Plants of floors 0 to 2 (left) and of floors 3 to 6 (right).

In a first step the results of in situ tests, carried out at the construction time, and of one drilling with continuous core were available. Another continuous core drilling was done in the second step and also the result of a SPT was available, made at about 200 *m* from the building. Four layers can be individualized in the subsoil, from the top: a surface layer of about 6 *m*, with negligible resistance to penetration; a non-cohesive layer up to 15 *m*; a cohesive soil up to 41 *m*, where a gravel deposit is present. The pile head is at 4 *m* from the campaign level and so is the average groundwater level. From the seismic point of view the subsoil can be classified as soil type C, with seismic wave velocity $Vs_{30}$ between 180 *m/s* and 360 *m/s*.

The geometry of the structure was known from the original architectural and executive structural drawings. Furthermore, detailed *in-situ* tests were carried out, so a full knowledge level for the structure was reached and, therefore, no reduction of the average material characteristics obtained from the experimental analyses was considered.

Most of the carrying structure was realized with steel of Fe360 type, with a yield strength $f_{yk}$ = 235 $N/mm^2$. The tensile tests carried out on a suitable number of specimens pointed out an average strength $f_{ym}$ = 268 $N/mm^2$. The reinforcing bars for concrete are Aq60-70 type, used at the construction time, having a tensile strength between 600 and 700 $N/mm^2$. The laboratory tests made on specimens extracted in site gave an average strength of 461 $N/mm^2$. The compression tests on concrete specimens pointed put an average cube strength $R_{cm}$ = 28 $N/mm^2$. The corresponding Young's modulus is $E_{cm}$ = 28300 $N/mm^2$.

The analysis under static loadings, carried out considering the various loading condition of the Italian code, gave positive results both for the superstructure and the foundations.

## 3 EXPERIMENTAL DYNAMIC ANALYSIS

An output-only strategy with ambient noise excitation was used for modal identification. The instrumentation was composed by 24 short-period seismometers (Kinemetrics SS-1, period = 1 sec), wired to a digital acquisition Kinemetrics Granite with a 24 bit A/D converter, able to simultaneously convert all the 24 channels. Four different sensor configurations were implemented, designed to identify the dynamic behavior of the structure. For the purpose of this paper only one of them (namely C2) was considered, which allowed analyzing the dynamic characteristics of the building. The positions of the sensors are plotted in 0. An acquisition lasting 30 *min* with a sampling frequency of 200 *samples/sec* was performed.

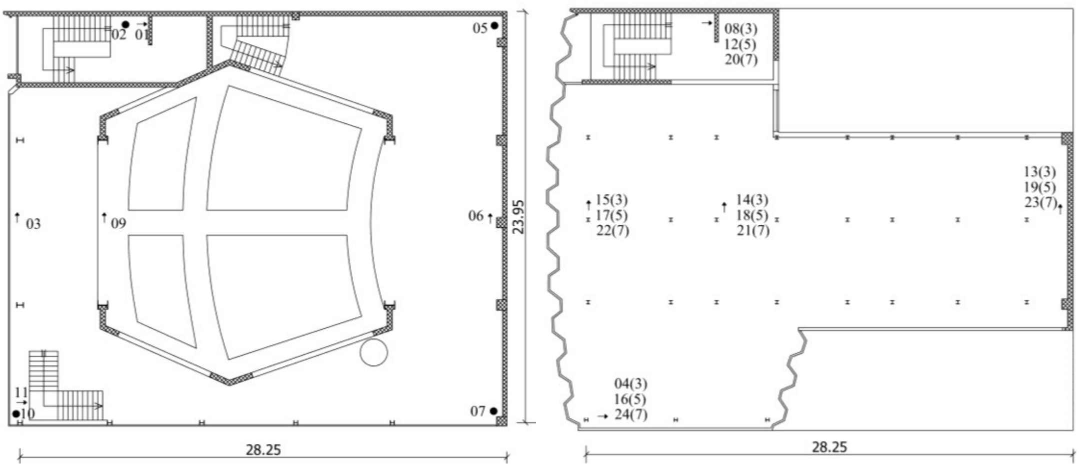



Figure 3 : Sensor configuration on floor 0 (left) and on floors 3, 5 and 7 (right, numbers in () indicate the floor).

The building showed a quite complex behavior mainly related to its irregular shape both in plan and elevation, but also to the presence of the adjacent building at the south side. The power spectral densities (Figure 4) pointed out the presence of two main resonance frequencies, equal to 1.67 *Hz* (0.59 *sec*) and 2.60 *Hz* (0.38 *sec*), respectively. The analysis of the cross spectra, in terms of amplitudes and phase factor, and of the corresponding coherence function, allowed stating that both the modal shapes associated to these frequencies showed rotational movements. In the first case (1.67 *Hz*) the rotation center is close to the S-E corner of the building, in the second one (2.60 *Hz*) out of the plant of the building in the N-E direction.

The other configurations confirmed the presence of the two modal shapes already described but showed also out of phase signals, probably due to a nonlinear behavior of the building. In one of them most of the sensors were oriented in the vertical direction. The results obtained demonstrated that all the points moved in phase and with the same amplitude, and allowed to exclude the presence of rocking effects in the building. This occurrence guaranteed the good performance of the pile system at least under permanent loadings and ambient vibrations.

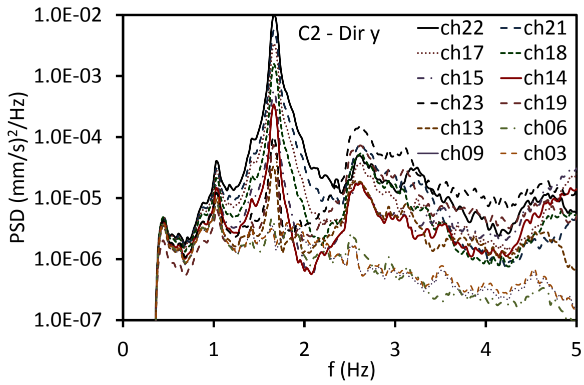



Figure 4 : Power Spectral Densities (PSD) of all the recordings in y direction (C2).

The results of the experimental static and dynamic analyses were used to update the finite element model set up in the first step and based on the original design. The masses related to the dead loads and to a fraction of the variable loads were considered. The structure was modelled by using frame and shell elements.

A good correspondence with the experimental results was obtained only considering the presence of the adjacent building, which, under ambient vibrations, certainly contributed to the dynamic behavior of the structure. This was accounted for in the model by means of horizontal springs, whose stiffness was calibrated in order to obtain the coincidence of the first experimental periods of vibration, equal to 0.58 *sec*, and the associated modal shape (Figure 5). The numerical analysis also revealed the presence of vertical modes, related to the presence of the theater room. This occurrence had been pointed out also by the experimental analysis.

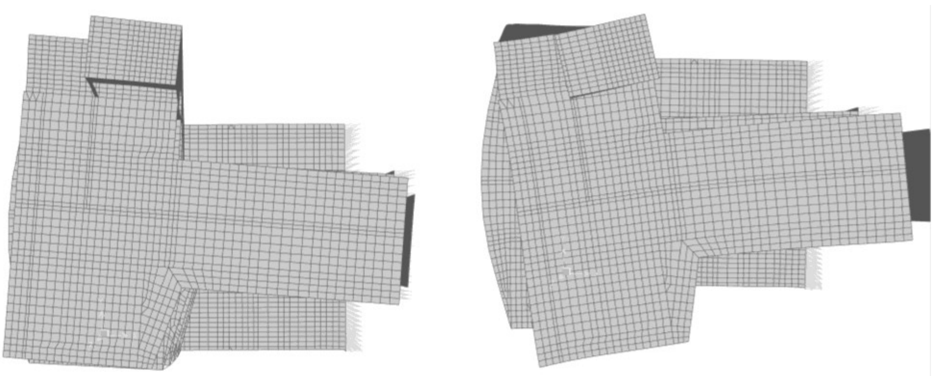
For the model of the building alone, set up just eliminating the horizontal springs, the fundamental period of vibration was about 1.0 *sec*. The seismic analysis was carried out by using the Ritz method that allows reducing the computational burden due to the very high number of degree of freedom and obtaining a participant mass higher that 90% with 50 modes.

## 4 SEISMIC VULNERABILITY ANALYSIS

As pointed out by the experimental investigations, the types of beam-column connections cannot guarantee a dissipative behaviour. So the seismic analysis was performed as for the static actions, assuming a behaviour factor $q = 1$. The response spectrum analysis was carried out with reference to the spectrum relative to the exceedance probability of 2%. The results were used to individualize all the elements that reach the elastic limit and the corresponding spectral amplitude. The comparison between these amplitudes and those of the elastic spectra relative to the various exceedance probabilities $P_{NCR}$ in 50 years allowed describing the vulnerability of the building.

| Mode | Experimental Periods (sec) | Numerical Periods (sec) | Periods without the adjacent building (sec) | Displ. |
|---|---|---|---|---|
| 1 | 0.59 | 0.58 | 1.01 | *Horiz.* |
| 2 | 0.38 | 0.28 | 0.44 | *Horiz.* |
| 3 | | - | 0.34 | *Horiz.* |
| 4 | . | 0.25 | 0.25 | *Vert.* |
| 5 | . | 0.19 | 0.20 | *Horiz.* |
| 6 | . | 0.19 | 0.19 | *Vert.* |
| 7 | . | 0.18 | 0.19 | *Horiz.* |

Table 1 : Periods of vibration of the modes with highest participation mass.



$T_1$ = 0.58 *sec* $T_2$ = 0.28 *sec*

Figure 5 : First two modal shapes accounting for the presence of the adjacent building.

The elastic and the plastic analyses (both applicable in this case) were used [7]. It is worth reminding that the first one refers to the ideal stress $\tilde{\sigma}$ at the characteristic points of the cross section, the second to the function $\tilde{\Lambda}$ of the demanding moment (Figure 6 left).

In Figure 6 (right) the elements for which the check is not satisfied are pointed out. As one can see, the limit values, both in the elastic and plastic analyses, are overpassed in several elements. The analysis in *x* direction gave similar results but, as one could expect, the worst case is that of seismic action along *y* direction. For all the elements the checks are satisfied up to a PGA = 0.10 g (on rigid soil), equal to 43% of the reference one ($P_{NCR}$=2%), when for the first element (column) $\tilde{\Lambda}$ . This corresponds to $P_{NCR}$=21% in 50 years ($T_{NCR}$=207 years). If one refers to the acceleration peak on surface (PGA·S) the value for which the first is not verified is about 47% of the reference one, due to the reduction of the amplification factor S when PGA increases.

In the hypothesis of rigid connection with the adjacent building, the period of vibration becomes equal to about 0.58 sec and the spectral amplitude decreases. The situation is worse than in the previous case. All the elements are verified up to PGA = 0.08 g (on rigid soil), equal to 35% of the reference one ($P_{NCR}$=2%), when the first element (column) in not verified. This corresponds to $P_{NCR}$=33% in 50 years and $T_{NCR}$=125 years. It is worth noting these considerations do not account for the effects of pounding between the two buildings.

The comparisons of the internal forces of the shear walls with the interaction domains gave always good results. Bending and shear checks were performed for all the reinforced concrete beams. The capacity moments were always higher than the demanding moments.

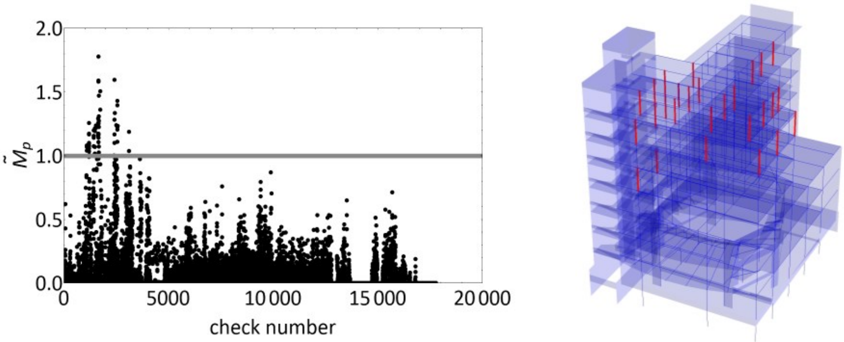



Figure 6 : Plastic checks (left) and steel members (right) that do not satisfy the plastic check for prevalent seismic action in $y$ direction.

## 5 NONLINEAR ANALYSIS OF THE PILE SYSTEM

The foundation are composed of concrete plinths, connected by means of concrete beams and supported by piles of about 40 m length (Figure 7). These have reinforcing bars only in the first 16 m. This unusual situation induced performing a detailed nonlinear analysis of the pile system.

The superstructure base shears and moments corresponding to the first modal shape, with prevalent motion in the $y$ direction, were applied at the top of piles and monotonically increased. The behavior of the piles was modeled considering the moment-curvature relationships under constant normal force. Three different cross sections were considered for each pile, due to the variability of the reinforcing bars, which are present up to the depth of 16 m. The corresponding moment-curvature plots are shown in Figure 8 for a given pile.

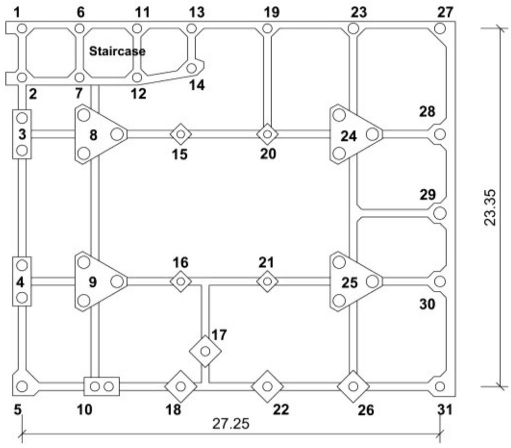



Figure 7 : Scheme of the foundation plinths with pile.

The soil was modeled by means of nonlinear springs, with stiffness deduced by means of the well-known *p-y* method. For the non-cohesive saturated soil, present up to $z = 15$ $m$, the relationships *p(y,z)* suggested by Reese et al. [9]**Errore. L'origine riferimento non è stata trovata.** were adopted; for the cohesive saturated soil, present from $z = 15.0$ $m$, the relationships suggested by Matlock [10] were used, but the contribution of this layer resulted negligible in practice. The model was developed in SAP2000 for the whole foundation. As previously said, modal base forces derived from the superstructure model were applied at the top of the piles, which were supposed to be connected by means of a rigid horizontal diaphragm.

The wall at the south side influences very much the forces distribution and the piles under that wall are much loaded. Bending moments along some piles at three different load steps are reported in Figure 9. They show the increasing moment up to the formation of the plastic hinges at the top and the unloading when the ultimate rotation is reached. The horizontal acceleration, given by the ratio between the horizontal force and the mass of the building, is plotted in Figure 10 versus the displacement of the control point, i.e. the point where the force is applied. The analysis was carried out in two ways, controlling the displacement and the force, respectively. The value of pseudo-acceleration equal to about 0.17 *g* can be assumed as limit one for the elastic behavior of the pile system.

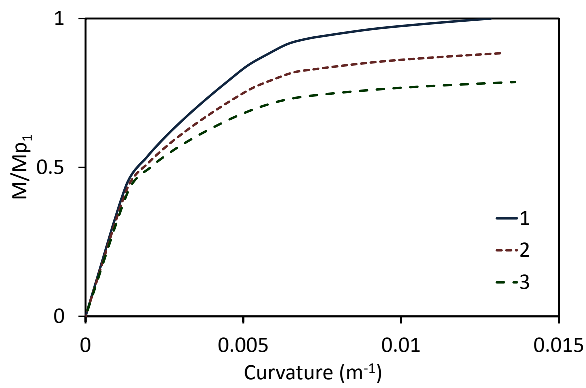



Figure 8 Moment-curvature curves for the three different reinforcing steel sections (Pile n. 27, $M_{pl}$= 1021 *kNm*).

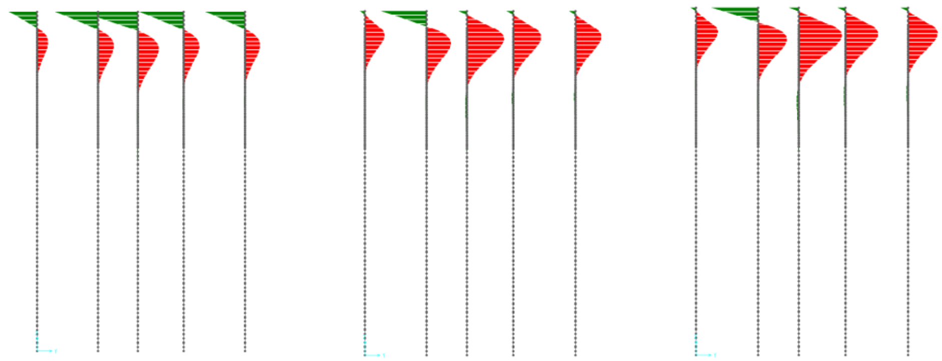


Figure 9  Bending moment in y-z plane at three steps for piles 27, 28, 29, 30 and 31.

## 6 BASE ISOLATION SYSTEM

A suitable solution, which can satisfy all the requirements, is the base isolation of the building. This allows reducing the seismic actions on the elevation structure and, as a consequence, on the foundation. A wider joint should be realized in this case but this does not represent a problem. In fact the joint should be realized in any case, substituting the south side walls with a new braced steel frame displaced of the needed gap with reference to the adjacent building.

The spectrum relative to $P_{NCR}$ = 2% in 50 years was assumed for the design, with reference to a damping ratio $\xi$ = 15%. Elastomeric isolators (HDRB) and sliders (SD) were considered, whose deployment was chosen with the objective of optimizing the dynamic behaviour of the building, so torsional effects were limited. 15 SD and 16 HDRB were used as shown in Figure 11. The isolator stiffness ($K_e$ = 1250 *kNm*) was chosen in order to obtain a period of vibration of about 3.0 *sec*, which guarantees an adequate decoupling between the motion of the superstructure with reference to the motion of the soil and also a significant reduction of the acceleration in the structure, much lower than those corresponding to its elastic capacity. The elastomeric isolators can support a displacement of 250 *mm*, which is also the value for the horizontal gaps. The response spectrum analysis was performed, combining the seismic effects according to the Italian code.

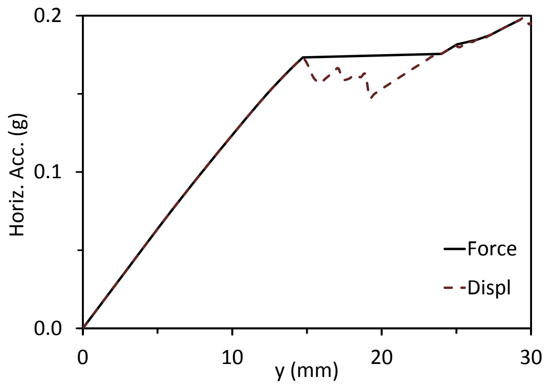



Figure 10 Static horizontal pseudo-acceleration in y direction versus displacement of the control point for the analysis carried out with force control (continuous line) and displacement control (dotted line).

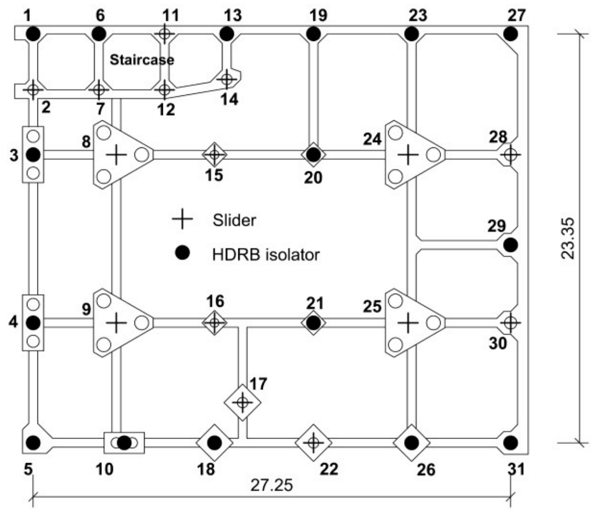



Figure 11 : Isolation system.

## 7 CONCLUSIONS

The analysis of an existing building, which could be used as strategic one, has been described in this paper. A finite element model was set up based on the original design, and the available experimental results were analyzed. Detailed experimental analysis on the materials and structure were carried out, and the dynamic characterization of the building was also performed by using ambient vibration tests.

The detailed and extended in situ experimental tests allowed assuming a full knowledge level of the structure so to avoid the lowering of the average material strength obtained experimentally. The checks under gravitational loads were satisfied both for the elevation structure and the foundations. The vulnerability analysis pointed out that the elevation structure safety was assured only up to seismic actions equal to 43% of those relative to the minimum exceedance probability of 2% in 50 years. For the foundations the situation is even worse.

The seismic isolation seemed to be the most appropriate retrofitting solution. It allows a significant reduction of the seismic action on the superstructure and the foundation so that to eliminate any structural intervention on them, while the realization of a gap with respect to the adjacent building at the south side should be done in any case to avoid pounding with the adjacent building.

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