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SEISMIC ISOLATION ON EXISTING RC BUILDINGS: OVERVIEW OF SOME ISSUES AND APPLICATIONS TO CASE STUDIES

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Abstract

The paper presents an overview of some common issues related to the seismic isolation technique applied to existing RC buildings. At first, the issues considered are briefly described. Then, solutions on how to solve them are illustrated with reference to case studies on which the seismic isolation is applied. The interventions are to date only designed or in progress of being realized. All case studies considered are existing RC structures designed only for vertical loads with a non-ductile behaviour and located in Potenza, a city in the South of Italy in a high seismic hazard area.

Keywords: rubber device. friction pendulum, seismic isolation, seismic devices, retrofit, existing RC building

1. Seismic isolation principles

Nowadays, seismic isolation is a technique largely applied for protecting buildings against earthquakes, applied both in designing new buildings, and in seismically retrofitting the existing ones. It is aimed to reduce the lateral accelerations demand and, consequently, to reduce elements internal forces through the superstructure natural period elongation. This is obtained introducing, typically above the foundation, a disconnection consisting of seismic devices having a low horizontal stiffness. In this way a decoupling between the superstructure and substructure is obtained, with an increment of lateral displacement demand (Figure *1*).

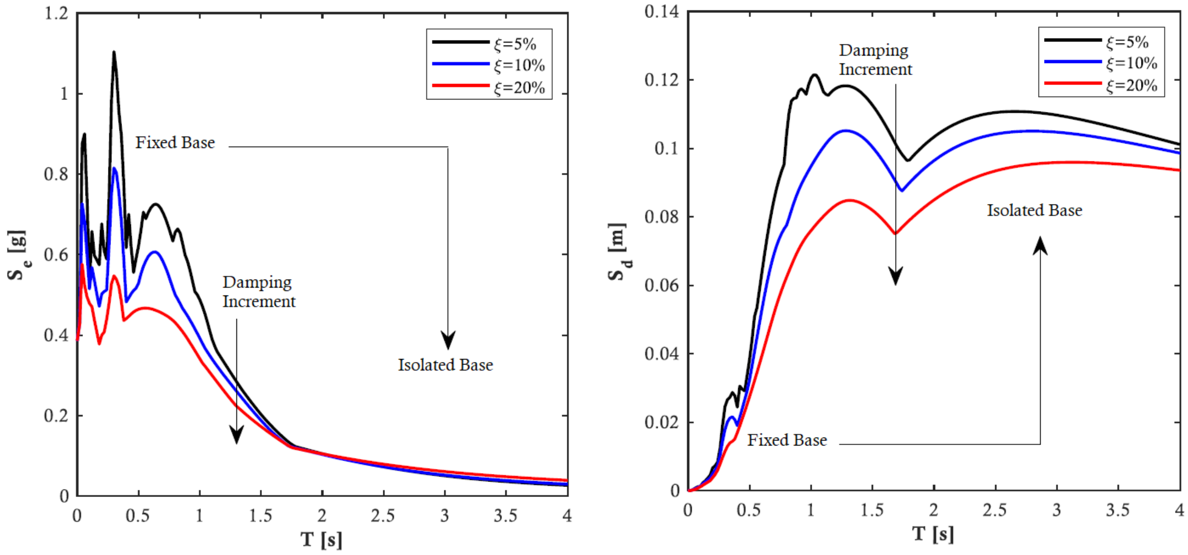

Figure 1: Acceleration and Displacement Response Spectra for different damping factor

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Very often the seismic isolation technique represents a very versatile solution in the case of Reinforced Concrete (RC) existing buildings, mainly designed only for vertical loads without any specific regulation for lateral loads, because of:

- it increases the safety building level since the seismic load acting on the superstructure may be significantly reduced due to natural period lengthening;
- it permits a more regular dynamic behaviour, reducing the eccentricity between the center of mass and stiffness;
- it ensures limited damages of non-structural elements and equipments for all the limit states, thanks to the drastic reduction of the interstory drifts and floor shear;
- it is minimally invasive since requires spaces of small dimensions to be realized, by acting on a limited portion of the structure, and in many cases it does not even require the occupants evacuation.

Starting from these premises, in this work some issues related to the seismic isolation strategy applied to existing RC buildings are examined. The issues considered play a central role during the seismic isolation design, since if not properly considered, they may lead to in an incorrect evaluation of the devices displacement demand and, in general, to a different behaviour of the isolated structure with respect to the predicted one. In detail, in this study particular attention is paid to:

- column cap and external jacketing, in order to guarantee a correct rebar anchorage length embedded in the concrete of substructures columns;
- torsional effects on the isolation system, to correctly evaluate the displacement demand;
- second-order effects, to consider the effective loads on the superstructure and substructure;
- superstructure stiffness evaluation, to properly design seismic isolation system.

At the first, the above-mentioned issues are discussed from a design perspective. Then, they are commented in detail with particular reference to some case studies briefly illustrated in this work.

2. Design Criteria

In this section, the issues previously listed are briefly introduced and commented from a design perspective. Afterwards, they are contextualized with reference to some case studies chosen in this work, where the interventions are yet in progress or, else, already realized.

2.1 Column cap and jacketing

A fundamental issue linked for installing seismic devices in the case of existing buildings, is represented by the cutting of existing RC columns involving, of course, as well the cutting of longitudinal steel rebars. For instance, Figure 2 refers to the case of seismic isolation realized at the top of the ground floor RC columns. In this new configuration, if no additional construction detail is adopted, the longitudinal bars at the superstructure columns base would have an inadequate anchorage length that, consequently, conspicuously reduces the columns flexural strength resulting hinged at the base. A possible solution is represented by realizing an appropriate column cap below the superstructure floor, in which longitudinal bars may be anchored. Whereas, in the case of seismic isolators installed above the foundation (seismic isolation at the base), an improvement of the column flexural strength may be obtained with an external additional column jacketing. Figure 3 depicts the solution proposed, where the floor at the base of the super-structure is realized through a concrete slab.

2.2 Torsional effects on isolated buildings

In designing the seismic isolation bi-directional earthquake loadings should be considered including the torsional effects in order to properly evaluate maximum displacement demand. This aspect becomes very important when seismic devices non-linear behaviour is modelled by means of visco-elastic equivalent schemes. As far as the torsional effects are concerned, caused by the structure geometry and the bi-directional earthquake loadings, recent studies on existing RC buildings showed that the displacement demand on devices may result greatly increased with respect to the simplified mono-directional analysis [1], resulting greater up to 1.8 times because of also the torsional effects produced by accidental eccentricity [2].

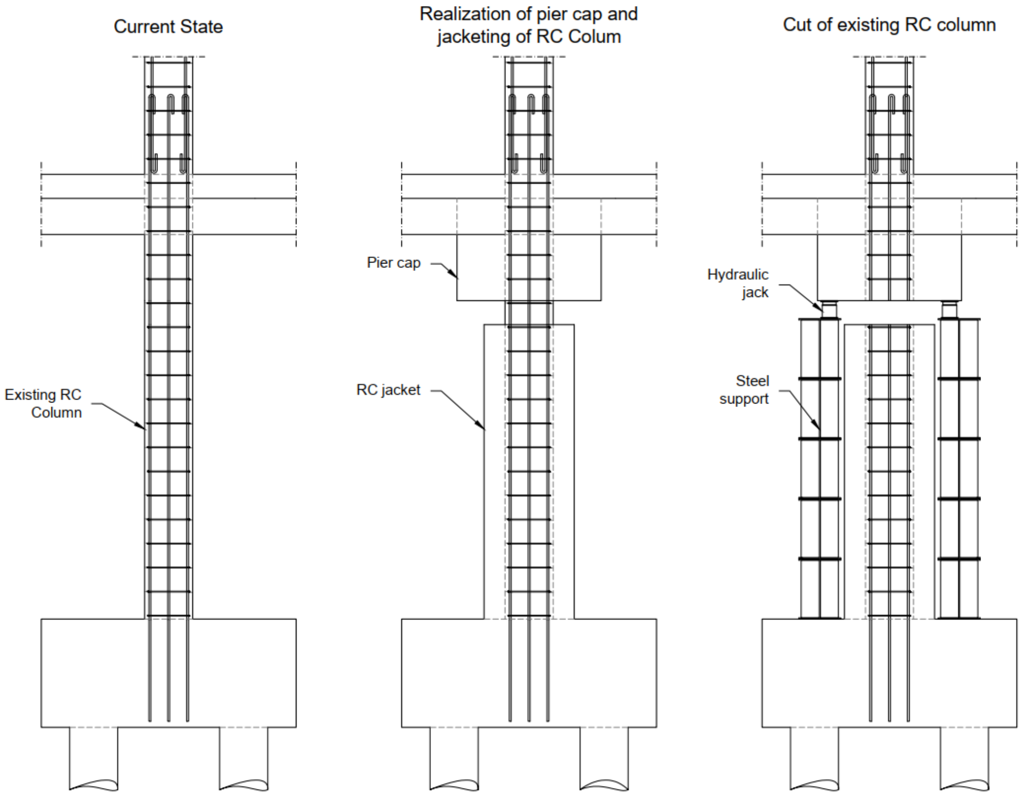


Figure 2: RC column cutting for seismic device installation

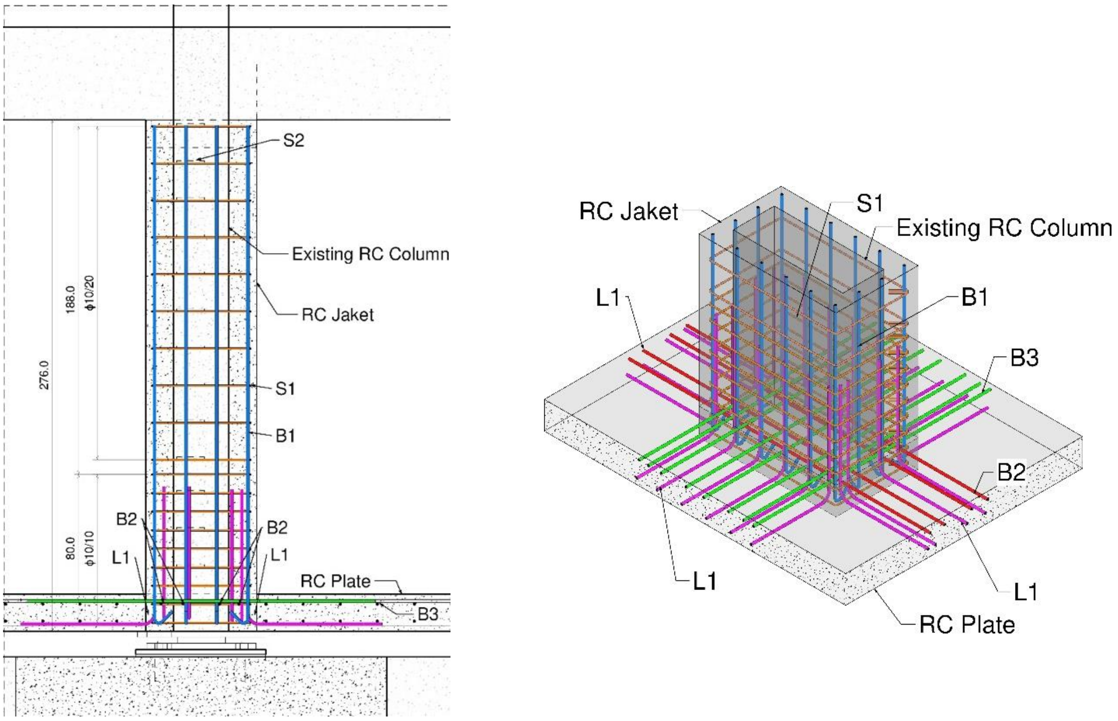


Figure 3: Example of RC columns jacketing with a concrete slab

2.3 Second-order effects

The installation of seismic devices within structures involves an in-depth analysis related to second-order effects due to load eccentricity acting during the earthquake motion. When the device is shifted, the vertical load does not act vertically and an additional moment is generated given, in the case of friction pendulum or HDRB device, by the axial load into half maximum displacement of the seismic device. This additional moment is acting on both superstructure and substructure with, of course, the same intensity. Whereas, in the case of sliding isolators, the axial load eccentricity provokes an additional moment depending on how the seismic device is mounted (Figure 4): if the plate is placed at the top of the substructure column (slider at the bottom of the superstructure), the additional moment acts on the substructure; conversely, the additional moment acts on the superstructure (the moment intensity in both the configuration discussed results of course the same).

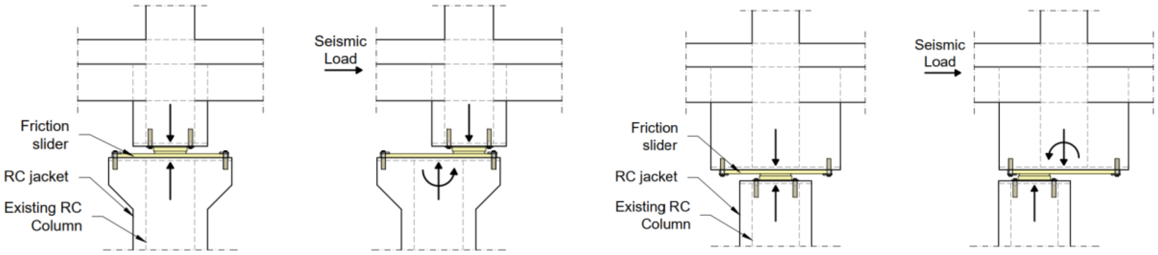


Figure 4: Sliding isolator: second-order moment due to the vertical load eccentricity

Therefore, the solution chosen may vary case-by-case, and it must be accompanied by a careful assessment of the second-order effects, including the local intervention needed. If the slider is applied on the superstructure or at the substructure e bottom column, the additional moment has to be supported by the superstructure beams converging at the upper column base where the plate is placed, requiring a local verification. Moreover, in order to reduce the internal forces acting on the column cap, the seismic device should be placed as low as possible for reducing the tensile forces acting along the horizontal tie according to the strut and tie model.

2.4 Superstructure stiffness

Superstructures stiffness plays a central role in the dynamic response of a seismically isolated building. Investigations carried out on existing RC buildings mainly designed only for vertical loads and seismically isolated highlight the importance to increase the superstructure stiffness with respect to the horizontal actions in order to reduce the effects of higher vibrational modes [3].

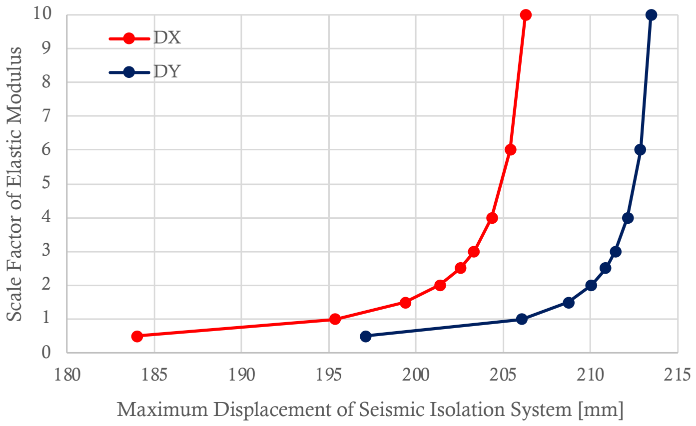


Figure 5: Displacement of seismic isolation system as a function of superstructure stiffness

Investigations conducted, linked to the FEM analysis, showed how the high buildings stiffness to the lateral actions is given by masonry infills (perimetral and internal), mostly interacting dynamically with the RC frame [4,5]. This important interaction is beneficial for seismic isolation because it increases the superstructure isolation degree (that should reflect a rigid-body behaviour), making the dynamic behaviour more regular, and strictly depending on the dynamic characteristics of seismic isolation system. The superstructure stiffening definitively implies an increment of the displacement demand on the seismic devices. As proof of this Figure 5 reports, by referring to a case study analysed, the isolation system displacement along the two directions without eccentricity by varying the scale factor of the superstructure elasticity modulus. As one may note, the higher the scale factor the higher the system displacement, with an asymptotic trend of the displacements as a function of elastic modulus.

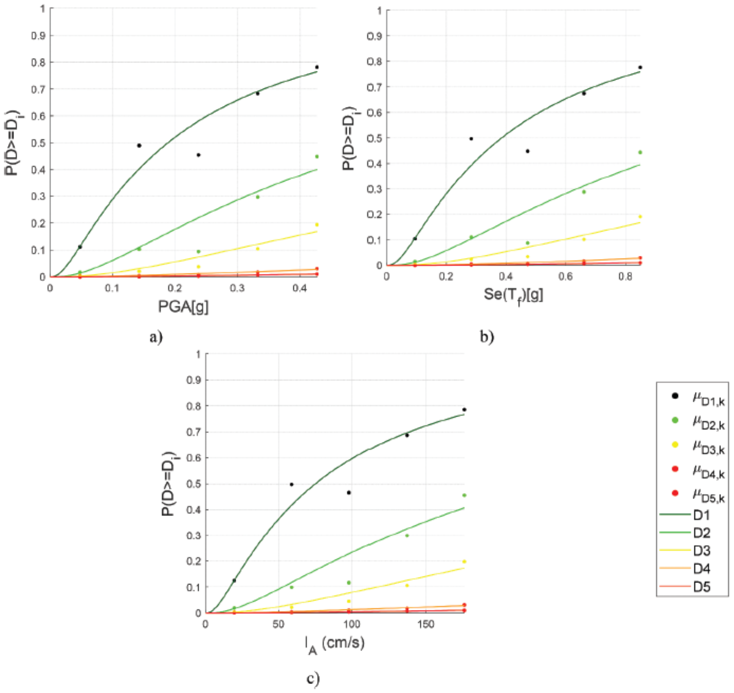
2.5 Fragility curves

Thanks to the isolation strategy the seismic damage of superstructure elements may be nullified due to a drastic reduction of interstory drifts and floor accelerations that are Engineering Demand Parameters (EDPs) strictly correlated to the elements' internal forces. Consequently, the internal actions of superstructures elements are mainly due to the vertical loads, that are the reference loads when existing structures were designed.

Benefits of seismic isolation may be proved through the fragility curves, expressing the probability to be equal or greater of a certain damage D_i for a given *Intensity Measure (IM)* representing the ground motion. In particular, one of the most largely applied functions to describe a fragility curve is the lognormal cumulative distribution function, expressed as follows [6]:

$$F_{D_j}(IM) = P\left(D \geq D_j \middle| IM\right) = \Phi\left[\frac{\ln(IM) - \mu}{\sigma}\right] \quad j = 0, 1, \ldots, 5, \tag{1}$$

where Φ is the standard normal cumulative distribution function of *ln(IM)* (i.e., gaussian function), where μ and σ are the natural logarithmic mean and natural logarithmic standard deviation defining the lognormal distribution, respectively. In this study, μ and σ of the *IM* considered are calculated using the maximum likelihood estimation method [7]. As example, fragility curves for existing RC buildings designed only for vertical loads are reported in Figure 6, by considering three different *IMs*, which are Peak Ground Acceleration (*PGA*), Intensity of Arias (I_A), and horizontal spectral acceleration [$S_e(T_f)$] [8].



a)

b)

c)

Figure 6: Fragility curves of RC buildings by assuming IM: (a) PGA, (b) Se(T_f), and (c) I_A [8]

3. Case Studies

The issues previously introduced are discussed in this section with reference to some Italian case studies. The interventions illustrated are designed or already in progress to be realized.

3.1 "Palazzo Gaeta" building

The building named "Palazzo Gaeta" was designed in 1962 only for vertical loads and then built in 1969. The RC structures consist of n. 12 floors: n. 8 levels above ground, including the roof, with a total height of about 24.60 m; and n. 4 underground floors, down to a depth of 14.30 m. In elevation the building has three distinct structures (hereinafter indicated as Fab. A, Fab. B and Fab. C) separated by structural joints. The total floor area is of about 1280.0 m^2. The structure is located on a slope with a difference along the height of about 14 m, where the Fab. A if located downhill. In the following some views of the FEM model implemented are shown (Figure 7).

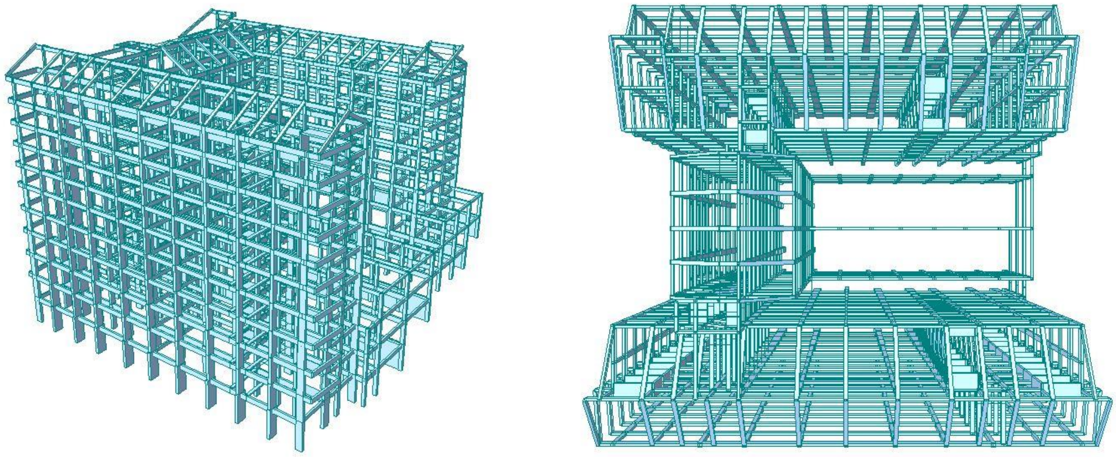


Figure 7: 3D views of the FEM model implemented

In this case an aspect that deserved particular attention in designing the seismic isolation system was the influence of the current lateral stiffness of the structures. In fact, the flexibility due to the height may involve, if not properly assessed, a not controlled isolation system response because of an inadequate seismic isolation degree. Therefore, in order to estimate the foundational periods of the structure, a dynamic identification was conducted starting from the results of a modal analysis carried out on the three frame structures (Fab. A, Fab. B and Fab, C). A comparison among the fundamental periods obtained along the two principal directions are reported in the Table 1. The periods estimated through the dynamic identification are indicated as T_{ID}, whereas the ones resulting from the modal analysis are indicated as T_{FEM}.

Table 1 – Fundamental periods comparison

Structure	**$T_{X,FEM}$ (s)**	**$T_{Y,FEM}$ (s)**	**$T_{X,ID}$ (s)**	**$T_{X,ID}$(s)**
Fab. A	1.6	2.8	0.5	0.6
Fab. B	1.8	1.6	0.6	0.5
Fab. C	1.2	1.9	0.4	0.5

As one may easily note, the fundamental periods obtained with the dynamic identification are significantly lower than the numerical ones computed with the numerical model, implying that the structures, in reality, are significantly stiffer with respect to the later loads than the ones implemented in the FEM model.

A plan of the hybrid isolation system adopted in this case is reported in Figure 8a, while Figure 8b reports its force-displacement relationship. Figure 9 shows a section of the building indicating that the isolation system is placed at three different heights.

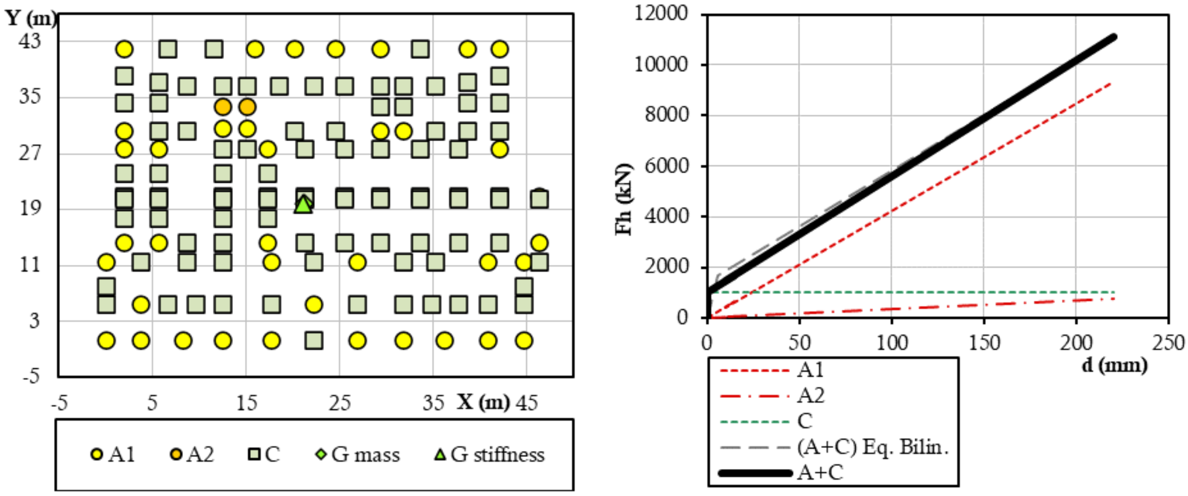


Figure 8: a) Plan and b) force-displacement relationship of seismic isolation system

In detail, the isolation system adopted has an equivalent viscous damping equal to 14.9% and consists of: n. 38 HDRB type A1 (28% of the total devices); n. 2 HDRB type A2 (1% of devices); and n. 94 friction sliders type C (70% of devices). The maximum displacement demand is of: 219.9 mm, calculated if only a SDOF system is assumed; 250.7 mm, including also the system eccentricity; 351.04 mm including both system eccentricity and torsional effects. As one may note a simplified approach considering as simple SDOF the superstructure would lead to a conspicuous underestimation of the displacement demand on seismic devices.

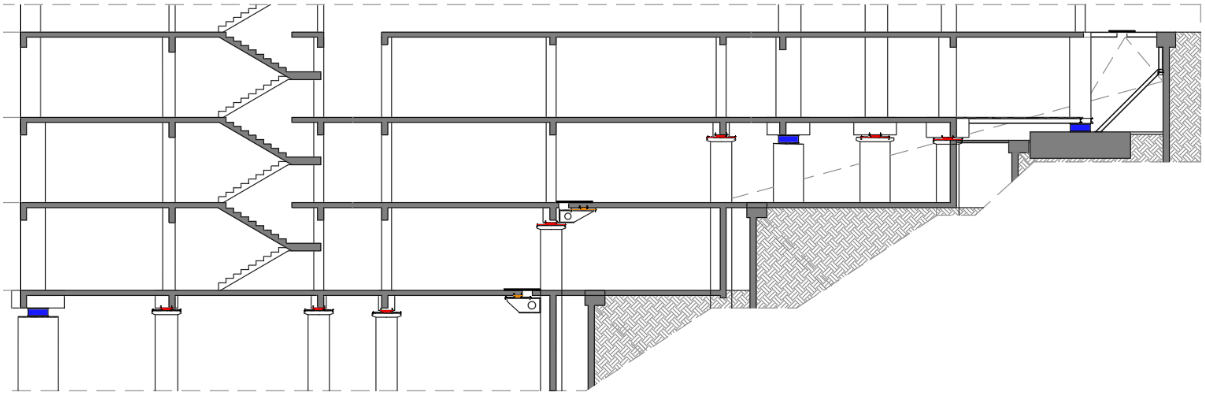


Figure 9: Palazzo Gaeta section illustrating the isolation system levels

3.2 "Zara 4" building

The building named "Zara 4" is a RC frame structure built in the '60s. It consists of n. 7 floors, including the roof, for the highest part of the building, of n. 5 floors for the lowest part, with a total height of about 21.40 m. In plan, the building may be schematized into n. 3 blocks, with a floor type of about 435.0 m^2.

Figure 10a reports the isolation system plan placed at the top of the ground floor columns, except for a small number of columns placed instead above the foundation. It is composed in total by n. 29 friction-pendulum devices, obtaining the force-displacement relationship indicated in Figure 10b. The maximum displacement demand is of: 163.1 mm, calculated if only a SDOF system is assumed; 197.0 mm, including also the system eccentricity; 275.86 mm including both system eccentricity and torsional effects. Again, a rough simplification as SDOF system would lead a significant underestimation of the devices displacement. For completeness, drift ratios evaluated for Life Satefy Limit State are reported in Figure 11. The ratios refer to: the As-Built configuration (i.e. fixed base building) and with the Isolation System. Moreover, in the same graph the limits for Damage Limit State in the case of fixed based building (dashed red line) and isolated building (dashed blue line) are reported according to the Italian Design Code [9]. As it is easy to observe the isolation system drastically reduces the drift ratios on the superstructure ensuring that RC elements and infills are not damaged with respect to the lateral seismic design action.

Figure 12 reports a detail of the columns jacketing designed for increasing the flexural strength and the cut elements rebars anchorage length. Whereas, Figure 13 shows a detail of connecting beams realized for resisting to the additional moments acting on the sub-structure due to vertical-load eccentricity occurs during the earthquake motion (second-order effects).

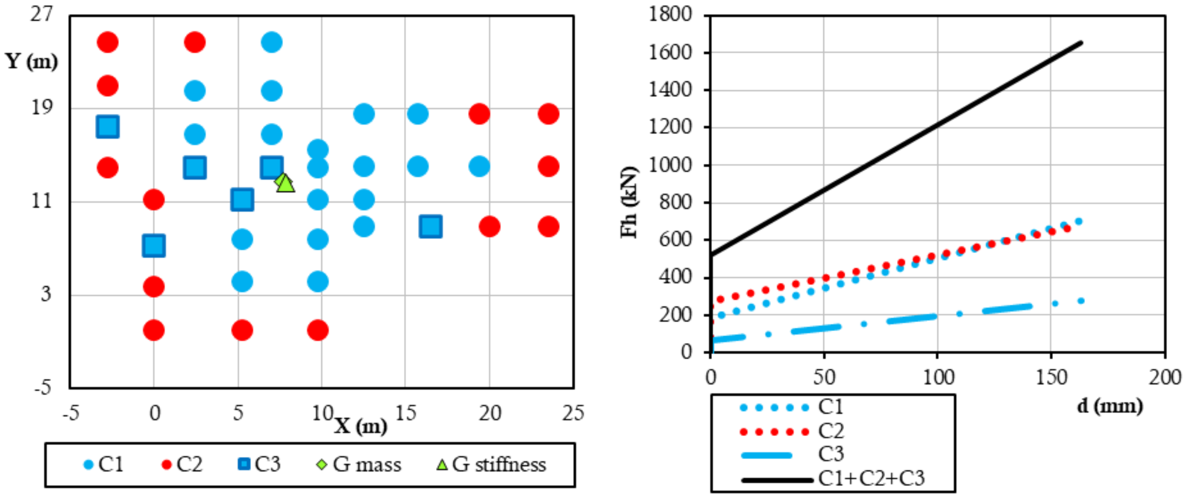


Figure 10: a) Plan and b) force-displacement relationship of seismic isolation system

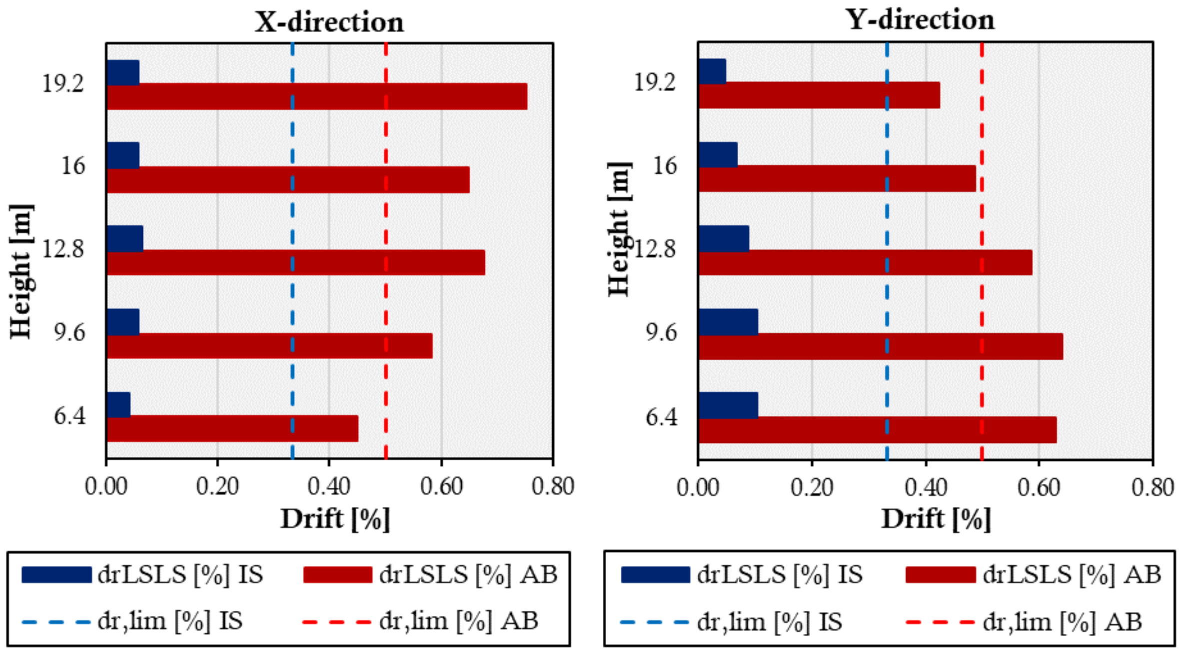


Figure 11: Drift ratios along the height: AB, As-Built configuration; and with the IS Isolation system

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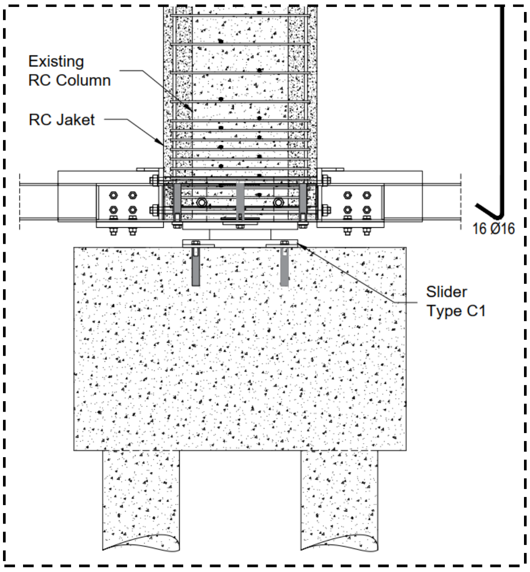

Figure 12: Detail of the external jacketing applied to the columns

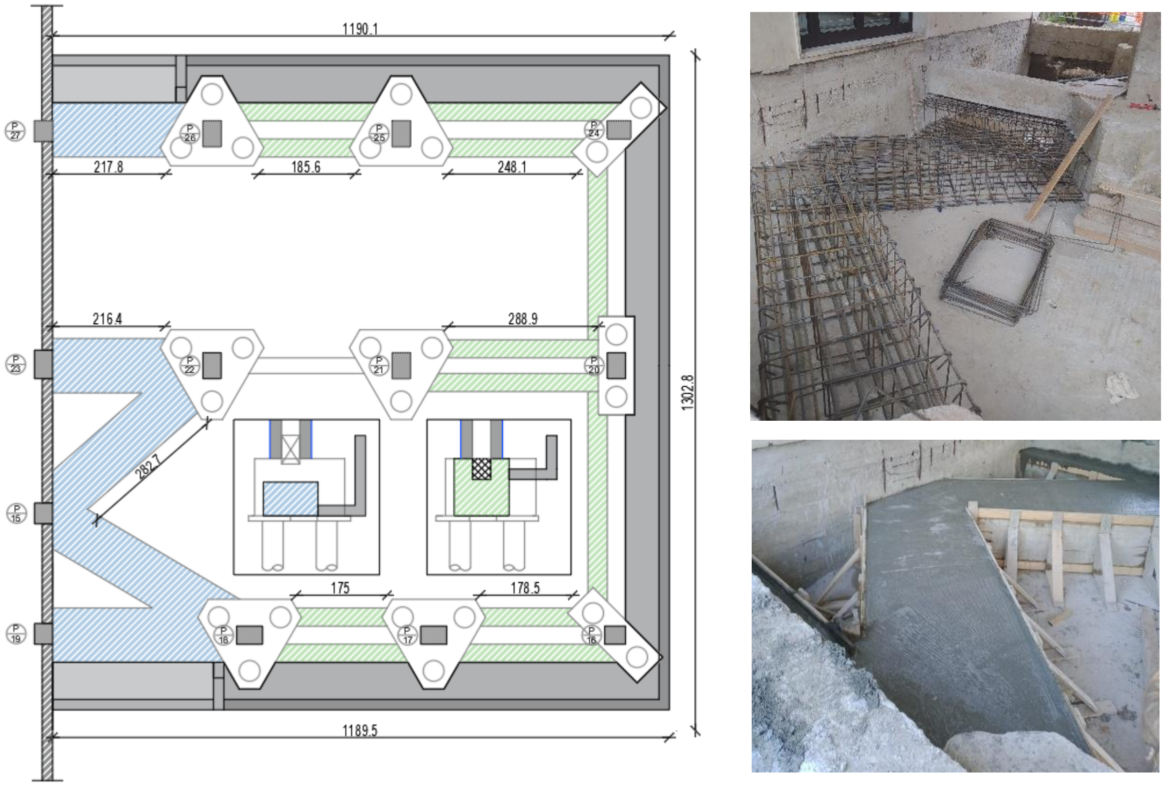

Figure 13: Detail of connecting beams realized among the foundation

3.3 “Zara 11” building

The building named “Zara 11” has a RC frame structure and it was built in 1969. It has n. 6 floors, with a total height of about 19.20 m. In plan the building may be schematically divided into three blocks with a floor plan of about 480.0 m^2. The hybrid isolation system is placed at the top of the ground floor columns, except for the ones below elevator and stairs that are at the base, above the foundation. The isolation system has 14.0% equivalent viscous damping and is composed by: n. 20 HDRB (type A), and n. 41 friction sliders (type C). The maximum displacement demand is of: 187.4 mm, calculated if only a SDOF system is assumed; 210.8 mm, including also the system eccentricity; 295.16 mm including both system eccentricity and torsional effects.

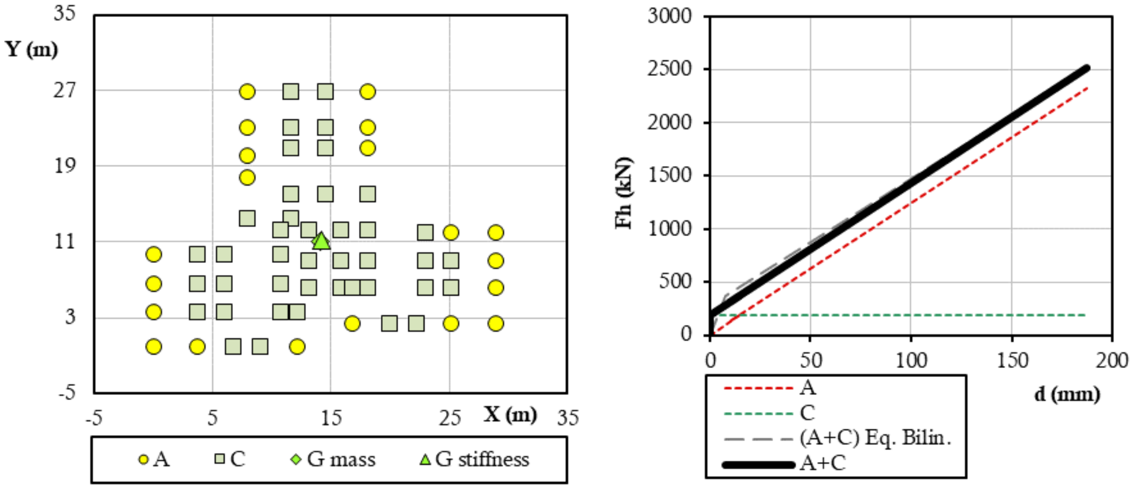

Figure 14: a) Plan and b) force-displacement relationship of seismic isolation system

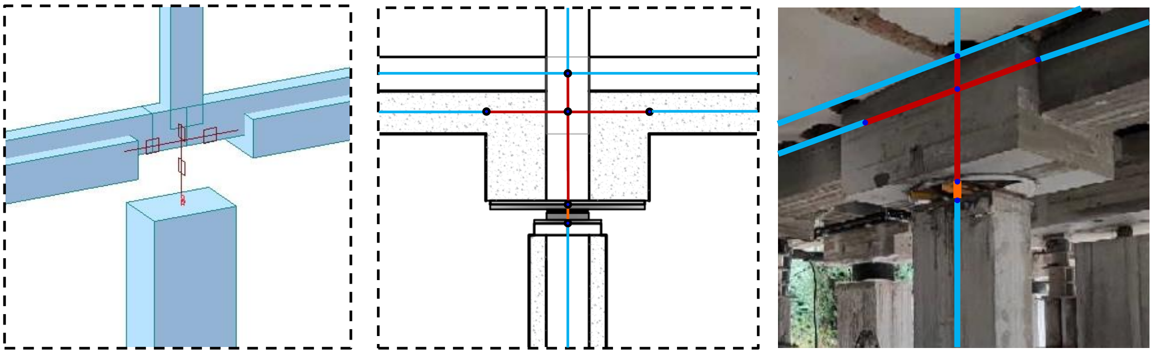

Figure 15: a) Plan and b) force-displacement relationship of seismic isolation system

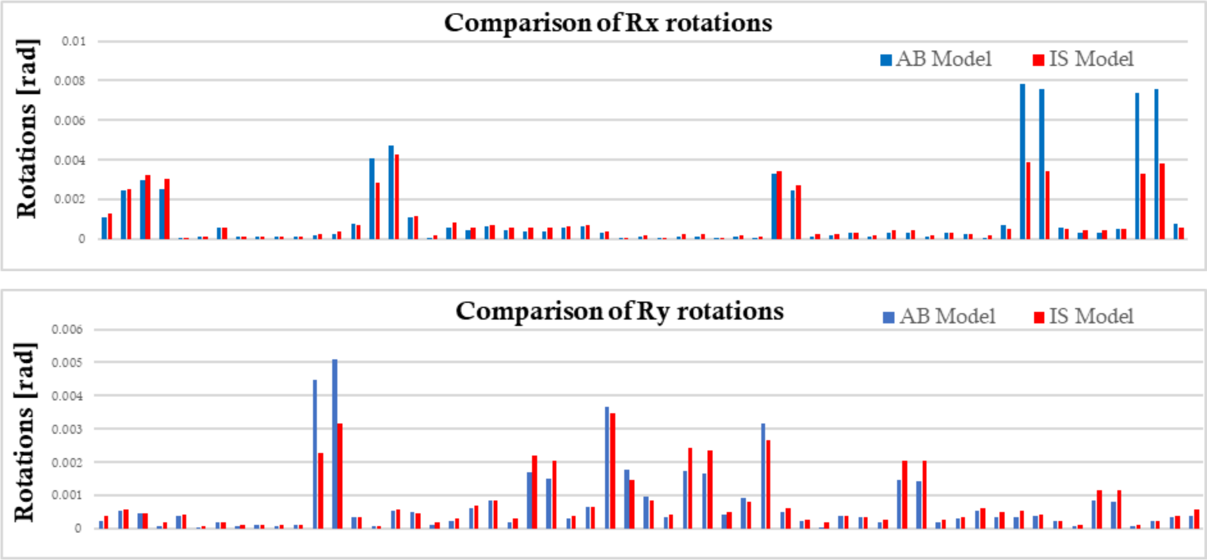

Figure 16: Rotation at the columns base without and with the grid floor along the two direction at the Life Safety Limit State

In order to reduce the additional superstructure internal forces (second-order effects) because of the HDRB and the slider devices shifted position, a floor grid has been designed at the isolation floor composed by new RC beams, jacketing of existing beams, and new steel beams, the latter inserted between the unconnected columns. The new steel beams may not be too high because of the installations, that are very close to the floor intrados. In order to quantify these additional internal forces numerical simulations have been conducted, reproducing the structural joints geometry including beams, columns and caps (Figure 15). Figure 16 compares the columns base rotations without and with the grid floor along the two directions at the Life Safety Limit State. It is easy to note that the grid added reduces the rotations of some columns and, consequently, the internal action born due to the second-order effects. Finally, Figure 17 reports some pictures of the intervention realized for installing the isolation system.

Figure 17: Details of the interventions realized

4. Conclusions

Seismic isolation is nowadays a largely applied strategy in designing new buildings or, else, in retrofitting the existing ones. However, its application requires the resolution of some specific aspects that may conspicuously condition the designing of the isolation devices and local details.

In this paper some issues related to the seismic isolation strategy applied to existing RC buildings have been discussed. The issues considered play a central role during the seismic isolation design because of, if not adequately considered, the may lead to an incorrect evaluation of the devices displacement demand and of the internal forces of the superstructure and substructure elements. Numerical simulations validate the importance of the issues investigated that very often are not negligible, and that may be properly taken into account for reaching the desired structure isolation degree, without any premature failure of seismic devices or structural elements.

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