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RINTC-E PROJECT: TOWARDS THE SEISMIC RISK OF LOW AND PRE-CODE SINGLE-STORY RC PRECAST BUILDINGS IN ITALY

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Abstract

This paper reports the preliminary results of a research project (RINTC-E) aimed at computing the risk of collapse in RC precast industrial buildings designed according to the codes in force in Italy in Seventies and Nineties. Companion papers describe the overall research project, funded by the Italian Civil Protection Department, its different areas of application (reinforced concrete, masonry, steel buildings, etc), and the overall seismic risk calculation procedure. This paper describes the design, modelling and pushover nonlinear analyses of one-story precast RC building, designed according to codes in force in Italy in Seventies. The structural nonlinear behavior is modeled using a lumped plasticity approach and the beam-to-column connection is based on friction forces. Collapse of the building is evaluated considering two failure conditions: i) 50% degradation of the maximum base shear recorded on the pushover curve; ii) beam-to-column connection failure.

Keywords: precast concrete, existing buildings, nonlinear analyses, lumped plasticity models, pushover analysis, friction connection.

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1 INTRODUCTION

Great consideration is given to the seismic vulnerability of Italian buildings, since severe consequences have occurred after earthquakes in different areas of the country. Seismic safety of precast structures was an underestimated topic in last century, even if damage and collapse of these constructions cause huge economic losses; for example, productive activity stops in industrial facilities, because of damage. Furthermore, this kind of structures is frequently subjected to heavy interventions and intended use changes, which can produce an increase in seismic vulnerability, if not adequately designed.

Recent seismic events, in particular Emilia Romagna earthquakes in 2012 [1], highlighted the importance of taking into account the seismic behavior of RC precast buildings, pointing out a significant design inadequacy for existing structures under seismic loads. These events have given to the research community the possibility to improve their knowledge about precast structures, providing a lot of data and practical experiences. Ercolino et al [2] handled the seismic assessment of a precast structure in Emilia Romagna that showed severe damage after last earthquakes, consisting in significant rotation at the base of the columns along one direction and connection failure. The developed model is able to reproduce the seismic behavior of the assessed structure and it can be very useful for modelling exiting precast buildings with friction connections.

Structural response and failure mode of existing precast constructions under seismic forces denote several weaknesses, mainly due to the fact that a high percentage of precast RC buildings in Italy were erected in areas only lately declared as high seismic zones; for this reason, the adopted design procedure meets the requirements of a non-seismic (or low seismic) code, providing buildings with insufficient resistance and ductility. Column slenderness, effects of seismic input asynchrony, floor deformations, influence of seismic vertical components, beam-to-column connection based on friction and eccentricity between columns and beams represent the main problems, as also demonstrated in Magliulo et al [3]. Despite all these problems, few studies on the seismic vulnerability of precast structures were conducted. In 2015 Casotto et al [4] developed a seismic fragility model for Italian existing RC precast buildings, with variable geometry and materials; the damage state was defined performing non-linear analyses and comparing the maximum demand for each state to the structural capacity. In the same year Palanci et al [4] developed a similar study considering a set of 98 one-story precast buildings in Turkey; the resulting fragility curves allow the classification of precast building in three groups, according to the strength and the ductility capacity, in order to make risk assessment and loss estimation easier and faster. Some studies focused the attention on the frictional beam-to-column connection behavior, whose failure causes the instantaneous collapse of the structure, because of the beam loss of support. Therefore, the neoprene-concrete frictional coefficient plays a very important role in the seismic assessment of existing precast structures, in which dowel connections were often not provided. Demartino et al [6] analyzed two different models, the first elastic and the second rigid non-linear, in order to evaluate the influence of different parameters on the minimum frictional coefficient necessary to avoid sliding. Magnitude, epicenter distance and soil type, besides dynamic characteristics of the structure, are the most conditioning factors. Magliulo et al [7] carried out a wide experimental campaign aimed to develop formulas for the neoprene-concrete frictional coefficient evaluation; pulling tests gave the most relevant outcomes, relating friction to the axial load acting on the neoprene pad.

In the present paper a seismic assessment of single-story RC precast buildings, assembled in 1970s, is carried out through static nonlinear analyses. Six different structures are designed according to the Italian DM 30/5/1974 [8] and CNR 10012/1967 [9], for three different sites (Milano, Napoli, Catania) and for two different heights of the columns (6 or 9 m), considering the same soil type. The development of a three-dimensional nonlinear model is needed; the base of the columns is intended to act as a plastic hinge, according to a lumped plasticity approach. Nonlinear pushover static analyses are performed along both the orthogonal directions, in order to validate the model and identify the building capacity.

2 CASE STUDIES: DESIGN

The assessed buildings are designed according to Italian codes DM 30/5/1974 [8] and CNR 10012/1967 [9]. Such regulations do not take into account seismic loads and the design follows a deterministic approach according to the allowable stress design.

Three sites (Milano, Napoli and Catania) are considered. Figure 1 shows the layout of the case studies, in particular plan and section views are illustrated. The global geometrical features are the same for all cases: buildings present one bay in transversal direction and four bays in longitudinal direction; principal beam span is 15 m, secondary beam span is 6 m. For each site, two cases with different values of column height (6 and 9 m) are analyzed, highlighting the influence of column slenderness on the global structural behavior. The presence of a crane is modelled only in terms of mass and vertical force; brackets, supporting the crane, are located at 1.5 m from the top of the columns.

According to the structural typology, roof elements and beams are designed only for vertical loads (permanent and variable actions) whereas the design of columns takes into account the wind load and temperature variation also, which are the only horizontal forces acting on the structure in the design phase.

Roof covering is made up of double T prestressed elements, disposed one close to the other and joined through a concrete slab with a thickness of 5 cm. This system allows to consider a rigid behavior for the floor in its own plane. Double T elements are linked to the principal beams through steel pins and plates, bolted to the jointed elements, ensuring in this way a hinged connection. They are designed for permanent loads (slab, screed and waterproofing), live loads (1 kN/m^2 for accessible roof) and snow, evaluated according to the site altitude and the roof geometric characteristics. The selected sizing is reported in Table 1.

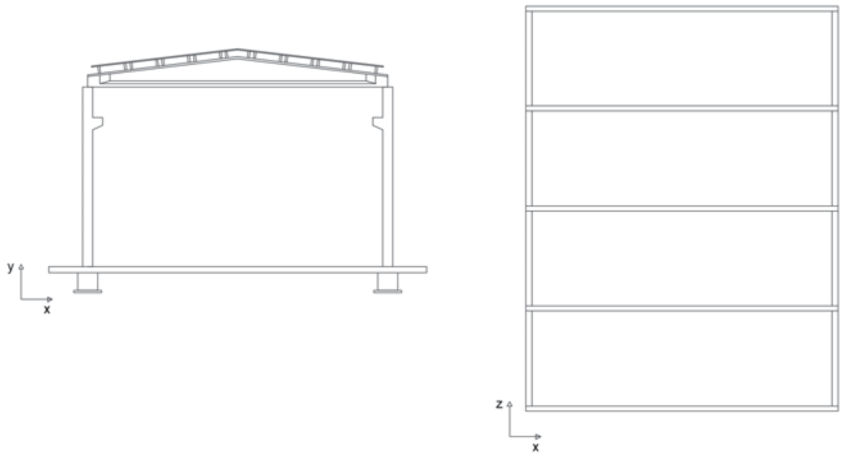

Figure 1 - Layout of the case studies: frontal view and plan view.

The principal beams present two kinds of variability: the first is a height variability, since the beam is higher in the mid-span than in the ends, and the second is a section variability, since the section shape changes gradually, from a T-section at the lateral sides to an I-section in the center part. This peculiar configuration aims to improve the beam structural behavior, increasing flexural and shear strengths where they are required. Because of all this variability, mean value of the height and a particular base dimension have to be considered in the nonlinear model (Table 1). This assumption does not affect the reliability of the results, since the assessed structural typology contemplates a nonlinear response under seismic forces only for columns.

Design of secondary beams is neglected, and pre-determined section dimensions are assigned to these elements (Table 1).

For both the covering elements and the beams the amount of steel reinforcement is not calculated because it does not influence nonlinear analyses.

Columns are monolithic precast square-shaped elements, assumed all equal in terms of section dimension and reinforcement quantity. According to DM 30/5/1974 [8], preliminary sizing is performed considering the columns subjected to a compression load and reducing the allowable stress by 30%.

Element	B[m]	H [m]	Site	H_{col} [m]
Roof element	1.60	0.40	All	All
Principal beams	0.25 (at the support)	1.14 (mean)	All	All
Secondary beams	0.30	0.50	All	All

Table 1 - Dimensions of the roof elements, the principal beams and the secondary beams

The acting axial load is given by the sum of the element self-weight, live loads, snow and the forces related to the crane (supporting beam and hook). Nevertheless, the preliminary sizing does not satisfy the verifications taking into account the second order effects; so the adopted section and longitudinal reinforcement ratios have to be increased for all the case studies. Figure 2 illustrates the final choices.

Socket foundations host the base of the columns allowing to consider a fixed restrain at the base.

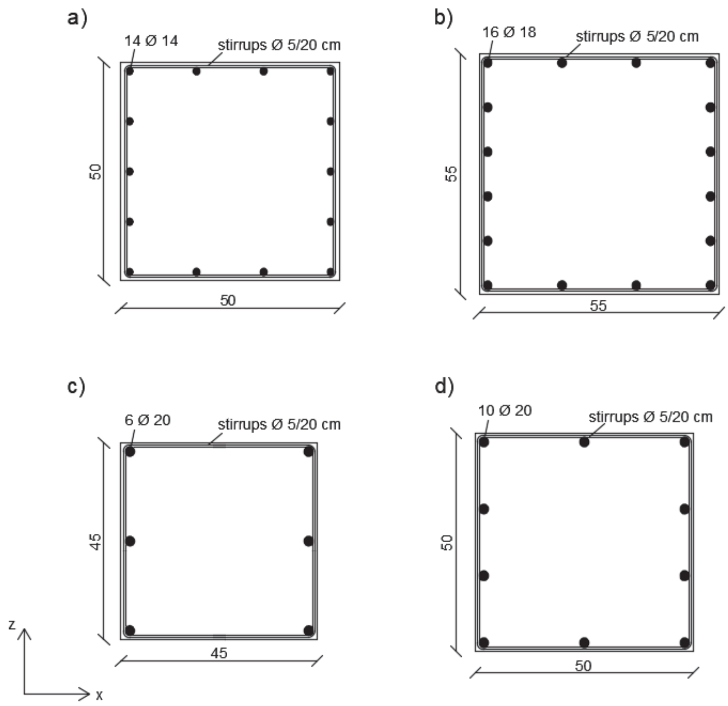

Figure 2 Column sections for a) the case of Milano with 9 m-height columns, b) the case of Catania and Napoli with 9 m-height columns, c) the case of Milano with 6 m-height columns and d) the case of Catania and Napoli with 6 m-height columns

The principal beams are simply supported on the columns through a friction connection, made with neoprene bearings. Bearing dimensions are chosen to be equal to 25x25x1 cm. Secondary beams, instead, are hinged to the columns with their connections through steel plates and bolts; mechanical connections are necessary due to wind actions. A neoprene pad is also placed at secondary beam-column interface, with dimensions equal to 10x10x1 cm, in order to allow a better stress distribution between the connected elements.

Such building configuration produces horizontal and vertical eccentricities between columns and beams, in both the principal horizontal directions. The value for these eccentricities are evaluated with the following formulas:

$$
\begin{aligned}
e_{h,pr} &= \frac{B_{col}}{2} - \frac{b}{2} \\
e_{v,pr} &= \frac{H_{bpr}}{2} \\
e_{h,\sec} &= \frac{B_{col}}{2} - 0.05m \\
e_{v,\sec} &= \frac{H_{b\sec}}{2}
\end{aligned}
\tag{1}
$$

where B_{col} is the base of the column, b is the side length of the neoprene pad, H_{bpr} is the section height of principal beams and H_{bsec} is the section height of secondary beams. Final geometric configuration is illustrated in Figure 3.

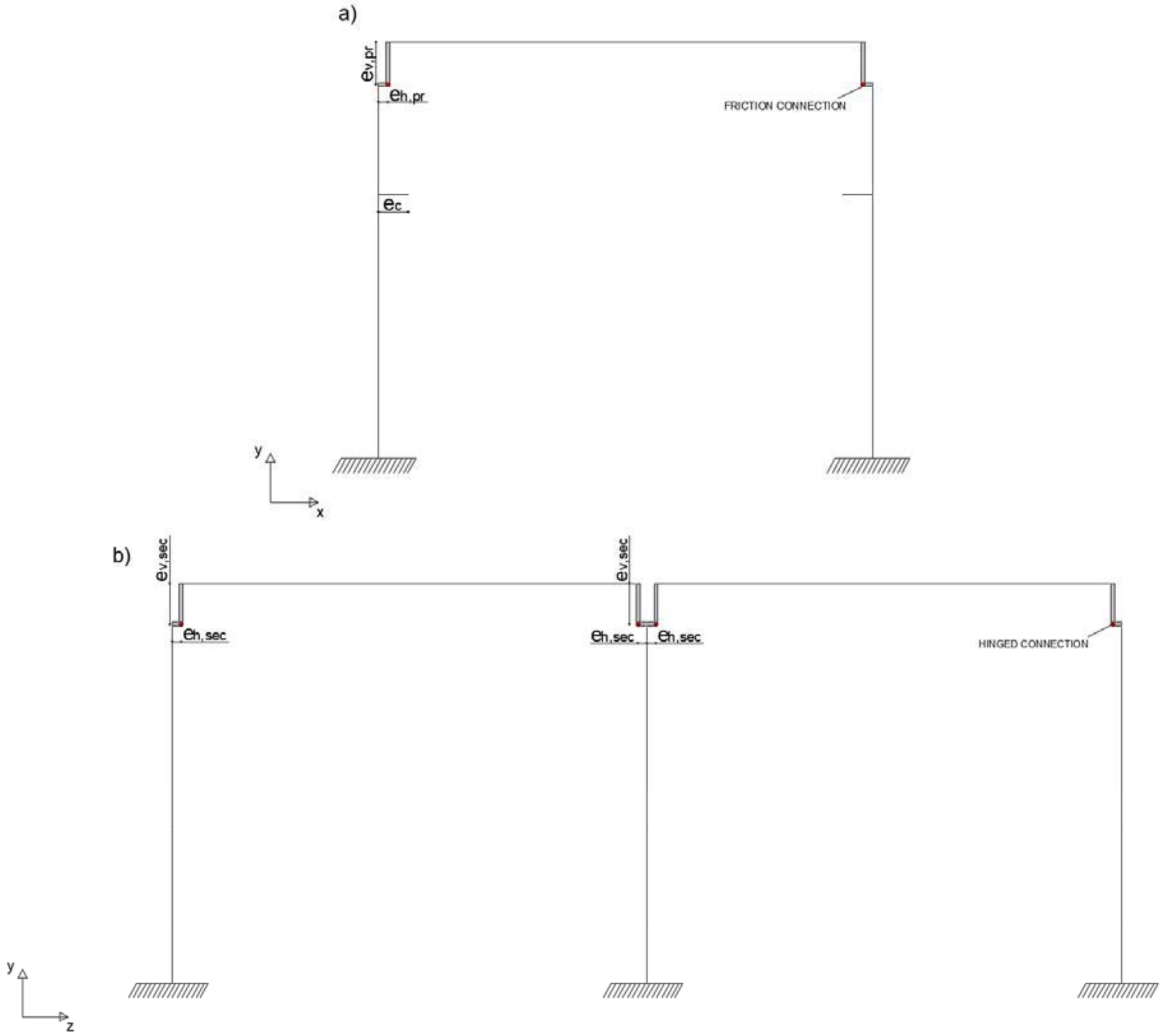

Figure 3 - Elastic model of the assessed structures: a) transversal direction and b) longitudinal direction

3 NONLINEAR MODEL

Nonlinear analyses require a modification of the elastic model illustrated in Figure 3. Plastic hinges are added at the base of the columns, in order to simulate the nonlinear behavior of the structure, according to a lumped plasticity approach. In these plastic zones, representing all the ductility resource of the structure, dissipation of seismic energy occurs. Hinges are modelled through a trilinear moment-rotation curve, fixing a yielding point, a capping point and an ultimate point. Values of bending moments and curvatures corresponding to the abovementioned points are estimated according to Fischinger et al [10]. Yielding moment is evaluated through an elastic fiber analysis of the column transversal section, considering three different materials: concrete core, concrete cover and steel. Unconfined concrete cylindrical mean strength is assumed equal to 42.96 MPa. This value is derived from the study [2], where the same type of structure with the same concrete characteristic strength was analyzed, and where some information were directly taken onsite. Indeed, a lack of knowledge can be denoted in the field of onsite testing of concrete belonging to RC precast structures built in the Seventies. For the material steel more data are available, based on extensive experimental campaigns on existing RC building steel bars, and the yielding mean strength results to be equal to 448 MPa. Yielding rotation is calculated according to Fardis, whose formulation allows to take into account the dependence on the shear span, and all the other parameters are estimated according to Haselton, since they do not depend on the shear span. The case of Milano with 9 m-height columns represents an exception to this validated procedure, demonstrating the lack of specific knowledge for existing precast RC structures modelling. Fischinger model [10] is inadequate for very high columns with a low amount of reinforcement, since it provides a very large value of yielding rotation and at the same time a too small value for the capping rotation. This produces a post-yielding stiffness larger than the elastic one, providing an unrealistic and unacceptable model. For this reason, only for this case, the monotonic behavior of plastic hinges is modelled completely according to Fardis [2]. Furthermore, as it can be found in the next paragraph, nonlinear static analyses show that collapses for all the assessed structures occur because of friction connection failure in the elastic field. Therefore, choosing Fardis approach seems to be reasonable since the model variation does not affect the elastic branch of the trilinear curve. Plastic hinges are different for corner columns and lateral columns, since the value of axial load acting on them changes according to the area loading the column. Furthermore, due to the design depending on vertical loads, column sections show different reinforcement along the two orthogonal directions; for this reason, two different plastic hinges are needed along x and z directions.

Concerning the hysteretic behavior, the Ibarra et al peak-oriented model [11], based on energy dissipation deterioration increasing with the number of cycles, is adopted. A normalized energy dissipation capacity represents the fundamental parameter; it depends on the axial load ratio, the ratio of stirrups spacing to column section dimension, the effective ratio of transverse reinforcement and the ratio between the value of the shear force in equilibrium with the maximum flexural strength and the shear strength. Figure 4 and Figure 5 show the resulting moment-rotation curves for the cases with 9 m-height columns. For the sake of brevity, the case of buildings with 6 m -height columns is not reported, but the trend of the curves is the same of the case of Napoli and Catania illustrated in Figure 4. Obviously, since for the sites of Napoli and Catania the buildings are identical, same curves are provided for both cases.

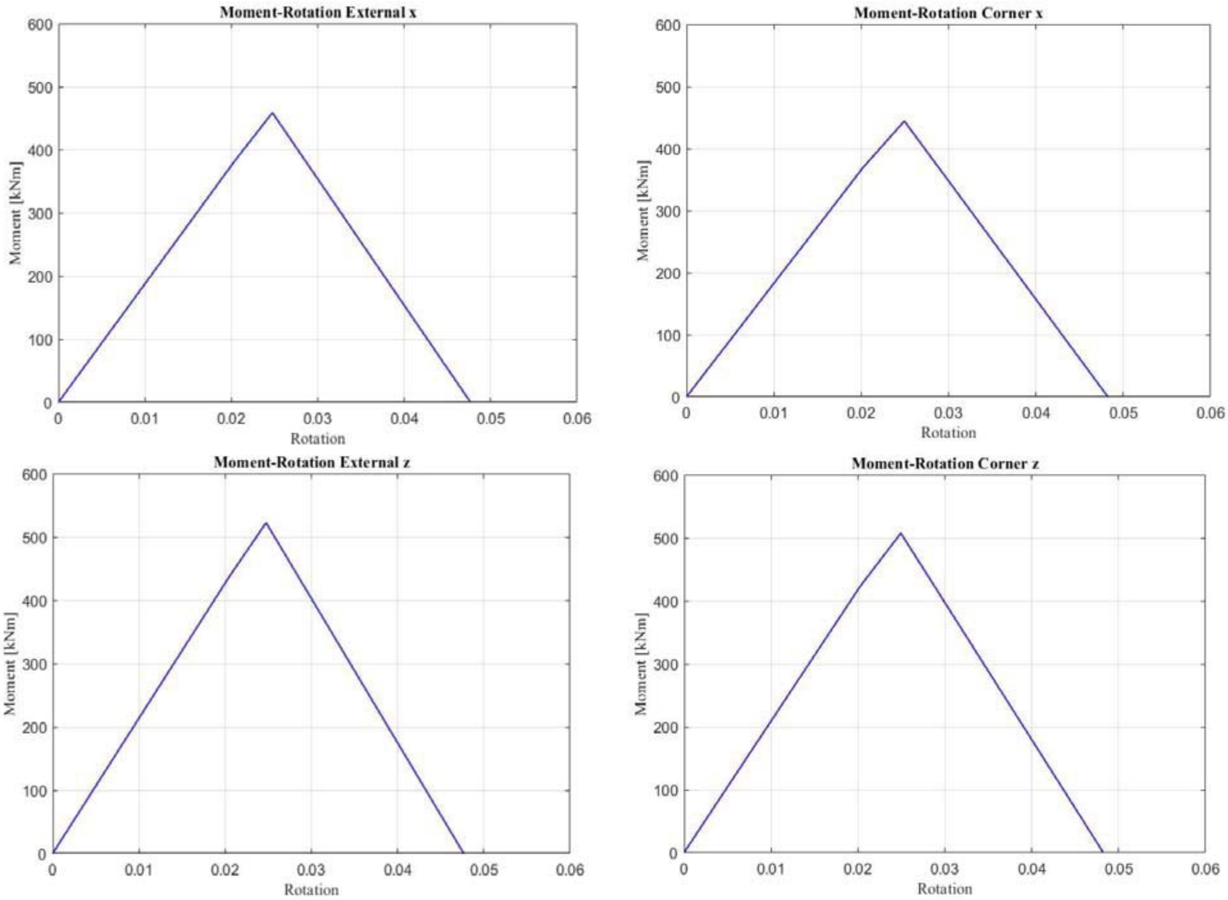

Figure 4 Plastic hinge moment-rotation curves for Napoli and Catania

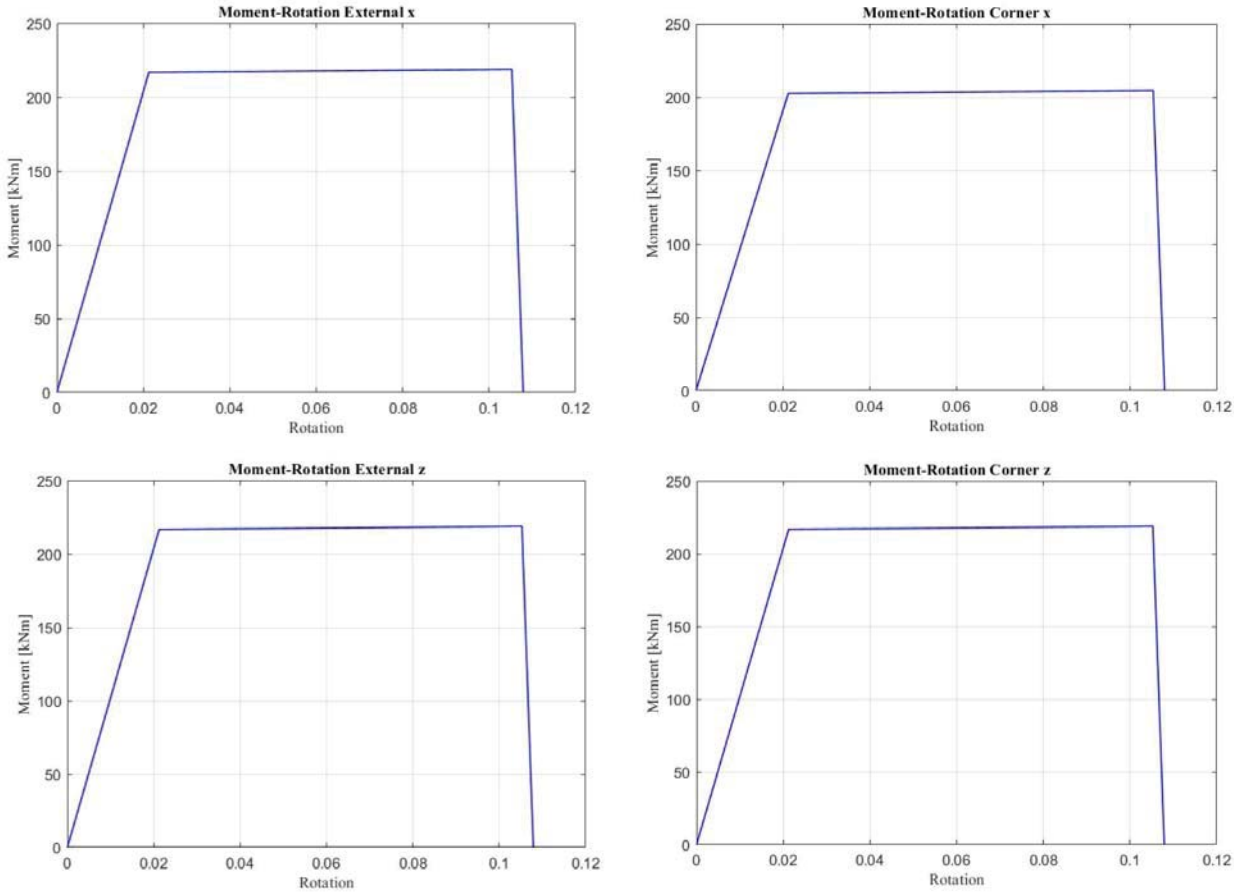

Figure 5 Plastic hinge moment-rotation curves for Milano

Modelling is carried out by means of the software OpenSees, implementing columns and beams as elastic elements and plastic hinges with the Ibarra deterioration model with a peak-oriented hysteretic response. Considering that columns and plastic hinges are connected in series, in order to provide columns with elastic behavior and to avoid numerical problems for plastic hinges, a small amount of elasticity is to be provided to plastic hinge, slightly modifying the column stiffness [12, 13]. Friction connections are implemented by means of flat slider bearing elements of OpenSees, which allow the translation in both the principal horizontal directions (Figure 6), at the reaching of a friction force, computed according to the Coulomb formula. Friction coefficient is found to be equal to 0.125, according to [7].

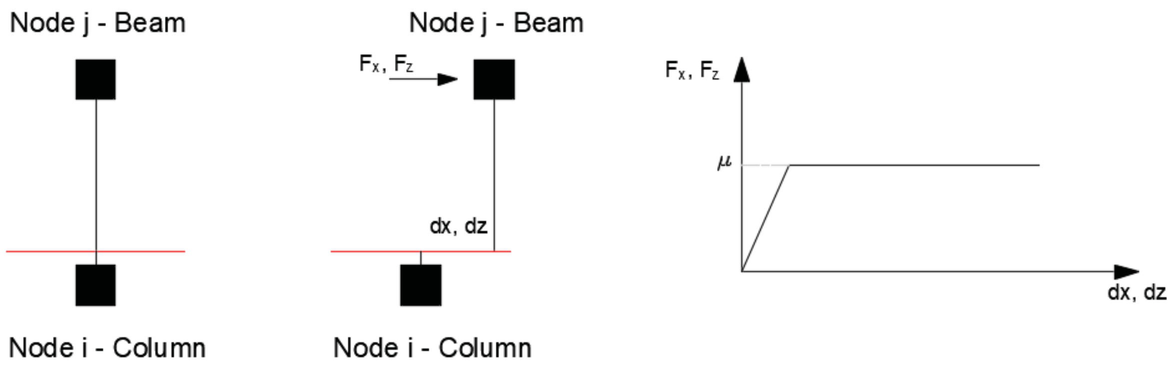

Figure 6 Flat slider bearing element, provided by OpenSees

4 ANALYSES OUTCOMES

Nonlinear static analyses are performed in order to evaluate the seismic capacity of the structures. This represents the first step for the vulnerability assessment, since the calculated capacity values are then compared to the seismic demand, provided by multi-stripe analyses through the application of 20 ground motions for each of the 10 intensity level, chosen for each site. Nonlinear multi-stripe analyses results are not included in this paper.

Nonlinear static analyses show that the collapse state is reached for all the structures in their elastic field, because of friction connection failure. Figure 7 and Figure 8 illustrate the resulting pushover curves, providing the building top center of mass displacement (assumed at the height of the mean axis of the main beams) vs the total base shear. Increasing the displacement, the force increases following the elastic branch of the capacity curve, up to the value of the friction force. At this point, beams start sliding upon the neoprene pad on column head, and a brittle failure occurs because of the loss of support of the beams, when the sliding displacement exceeds a certain value of displacement, depending on geometric features of the structures. The collapse displacement can be evaluated as the sum of two contributions: i) the column top elastic displacement under the friction force; ii) the sliding displacement before the loss of the support.

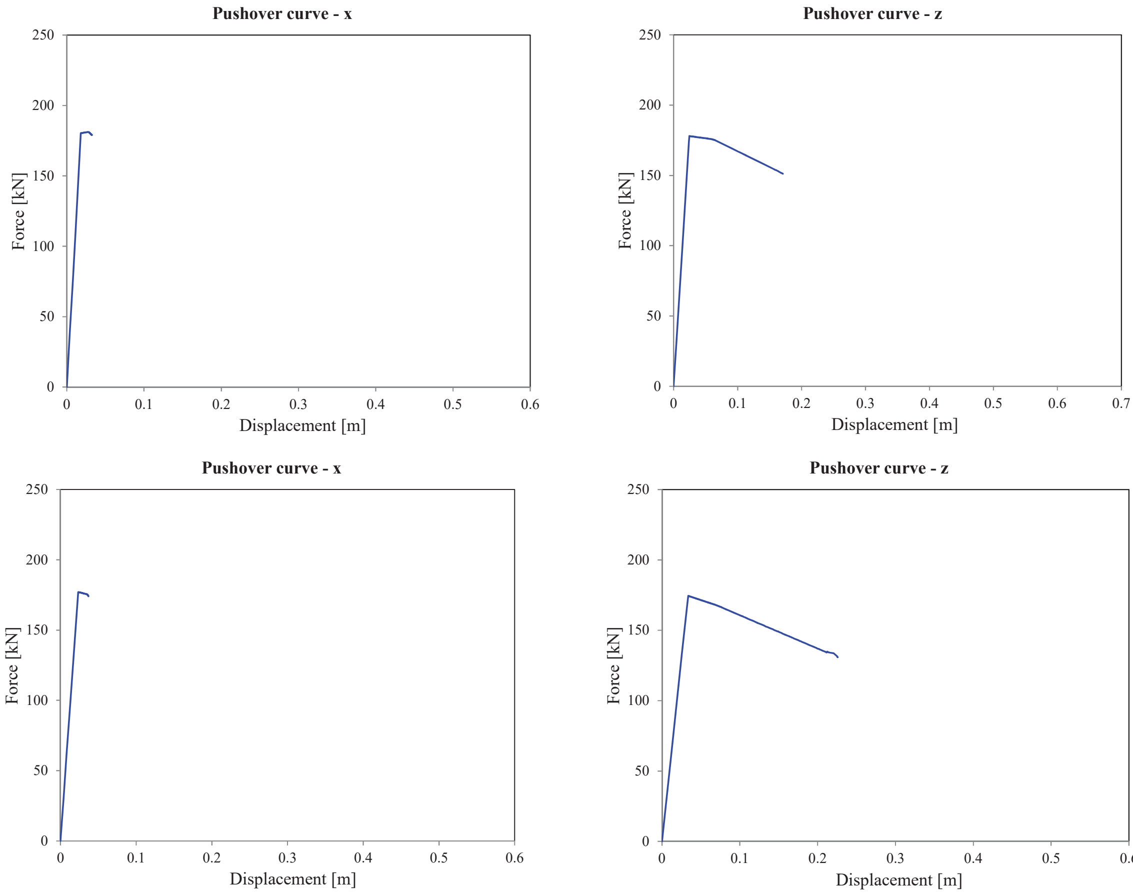

Figure 7 Pushover curves for Catania and Napoli (top) and Milano (bottom) in the case of columns 6 m high

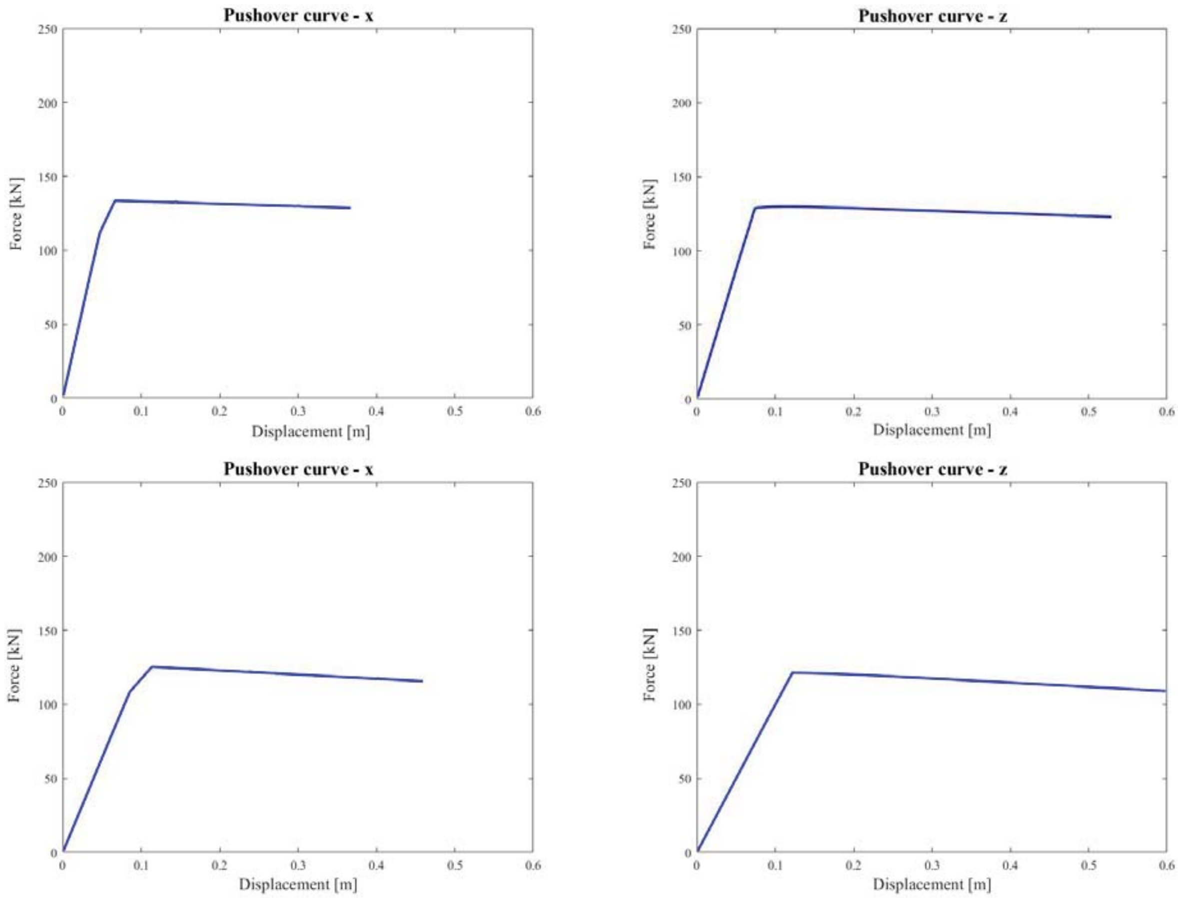

Figure 8 Pushover curves for Catania and Napoli (top) and Milano (bottom) in the case of columns 9 m high

5 CONCLUSIONS

The main conclusions of the research study shown in this paper are summarized in the following.

- Design according to Italian code in force in Seventies, in zones that were not classified as seismic zones, provides slender structures with a small amount of transversal reinforcement.
- Design according to Italian code in force in Seventies, in zones that were not classified as seismic zones, provides friction beam-to-column main connections.
- Further research is needed in order to model the nonlinear behavior of columns of existing single-story RC precast buildings.
- Seismic capacity of existing single-story RC precast buildings designed in Italy in Seventies, in zones that were not classified as seismic zones, is provided.

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