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INFLUENCE OF NONLINEAR MODELING ON CAPACITY ASSESSMENT OF RC FRAMED STRUCTURES

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Abstract

Many existing buildings in the world present serious seismic deficiencies and need to be retrofitted. However, the basis for an effective seismic retrofit intervention is a reliable assessment of the structure. To this end, nowadays structural engineers can simulate the response of structures subjected to earthquake excitation by nonlinear numerical models. These models consider explicitly the mechanical nonlinearities of the structural members, identify the parts of the structure where yielding takes place, quantify the demand of plastic deformation and force. Furthermore, a number of models is available to carry out the nonlinear analysis of structures. All these models are able to provide a detailed representation of the seismic response of the structure. However, they are controlled by many parameters that need to be properly set to obtain an accurate prediction of the response.

Based on the framework depicted above, the target of the Reinforced Concrete Work Package 2 of the ReLUIS 2018 project was to examine and compare different nonlinear modelling techniques used to evaluate the response of structures by pushover analysis. To this end, a case study building is analysed by the eight research units involved in the project by different nonlinear numerical models. The building presents very different lateral stiffness and strength in the longitudinal and transverse directions. Each numerical model is run two times including and not including the masonry infills. Furthermore, pushover analysis is run two times with forces in the longitudinal and transverse directions. Finally, the results are compared to illustrate advantages and limitations of each nonlinear modelling technique.

Keywords: Existing buildings, RC framed structure, infills, nonlinear modeling, pushover analysis, seismic assessment.

1 INTRODUCTION

In Italy, as well as in other earthquake prone countries, buildings were erected in the past without considering the effects of seismic excitation or according to obsolete seismic design provisions. Furthermore, many existing buildings suffer from significant structural degradation because of the original use of materials with low mechanical characteristics or the natural decay of their features. The vulnerability of the existing building stock is a serious economic and social concern in many countries and the need for retrofitting or rebuilding grows as time progresses and existing structures become older and degrade further. The precondition of any effective seismic retrofit intervention is a reliable assessment of the structure safety. A correct seismic vulnerability analysis should accurately detect the seismic deficiencies and quantify the seismic capacity of the structure. This target can be achieved by means of nonlinear numerical models and nonlinear methods of analysis, which explicitly consider the inelastic response experienced by the structural members. A great variety of nonlinear numerical models is available, each one presenting advantages and limitations. Furthermore, nonlinear numerical models require many parameters to be set. An improper choice of the numerical model or the incorrect selection of the model parameters may undermine the accuracy of the analysis.

Using the case study of an existing reinforced concrete frame structure, this paper analyses different types of nonlinear numerical models, their behavior and their response predictions. Both lumped and distributed plasticity models are considered. Phenomenological and fiber section models are used to simulate the nonlinear response of the cross-sections. The analyses are run by means of different computer programs by different research teams of the Reinforced Concrete Work Package 2 of the ReLUIS 2018 project. A FE model with shell elements is considered too. Two numerical models are built by each research team, with and without masonry infills. Preliminarily, the parameters that control the numerical models are set based on shared assumptions made to obtain results as homogeneous and comparable as possible (in terms of Base shear – Roof displacement relationship).

The results are used to detect the seismic deficiencies of the structure, to determine the collapse mechanisms and to evaluate the capacity of the structure (maximum base shear and roof displacement) the structure can sustain. Finally, the results are compared to illustrate advantages and limitations of each numerical model and to analyze the effects of the infills on the result of the seismic assessment of the structure.

2 CASE STUDY BUILDING

The case study building is derived from the De Gasperi-Battaglia school building in Norcia (Italy) shown in Figure 1. The construction of the building dates back to the early sixties. In 1997, the building was stricken by the Umbria–Marche earthquake and suffered significant damage. In 1999, a comprehensive structural survey was executed to serve as basis for the design of seismic upgrading interventions. The survey included the analysis of constructive details of non-structural elements, the collection and study of the design drawings and reports of the structure, the verification of the geometry of the structure, and the characterization of the mechanical features of materials by experimental investigation.

The building is approximately rectangular in plan, four storeys high, endowed with unidirectional hollow clay block-cement mix slabs, and protected by a pitched roof. Two Gerber joints separate the building in three independent blocks with RC framed structure. The left block (Fig. 1), which is the one analysed in this paper, is rectangular shaped in plan with maximum and minimum dimensions equal to L = 24.5 m and B = 12.2 m. The inter-storey height is equal to 3.5 m at the 1st storey, 3.3 m at 2nd, 3rd and 4th storey, and 2.1 m at the ridge of the roof. The structure of the analysed block is constituted by three seven-bay frames and seven two-bay frames arranged along the longitudinal and transversal directions, respectively. The unidirectional floor slabs rest on the beams of the frames arranged along the transversal direction and on the beam sustained by the adjacent block by means of the Gerber joint. The external frames are endowed with masonry infills constituted by two layers of clay bricks. The external layer of the infill panels is made with clay solid bricks of 12 cm thick, while the internal one is 8 cm thick and is made with clay hollow bricks. The masonry infills encased in the longitudinal frames are partial height because they are surmounted by windows that extend from column to column. Out of the transversal frames, only the one located on the left side of the block is infilled. In this case, the infills are full height.

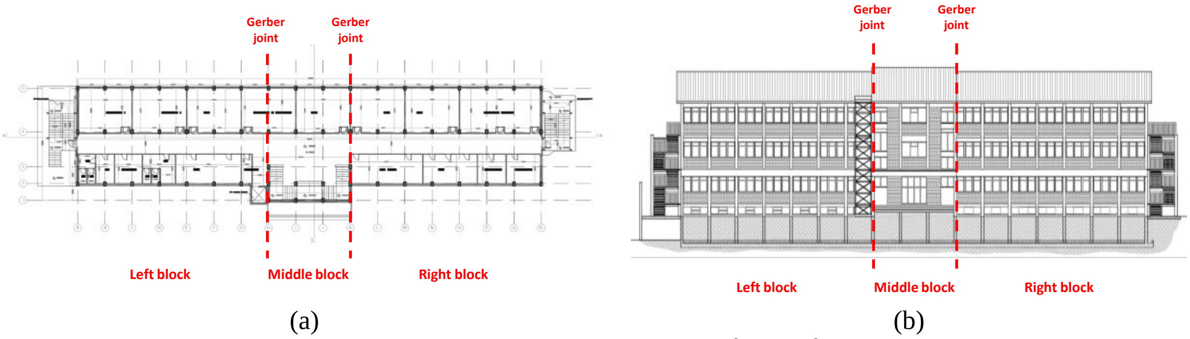

Figure 1. De Gasperi-Battaglia school building: (a) plan layout of 2nd and 3rd floor, (b) north front view

The analysis of the technical drawings provides the description of the structural and non-structural elements and in turs the data for the evaluation of the gravity loads. The characteristic values of the dead (g_k) and live (q_k) loads are listed in Table 1. The design drawings show also that the beams of the frames arranged along the longitudinal direction are provided with flat beams. Instead, deep beams are used for the transversal frames. Longitudinal reinforcement of the beams is made by bent-up bars in compliance with the design practice of the time. Rectangular cross-section oriented with their short side orthogonal to the plane of the transversal frames is used for all the columns. The size of the column cross-sections reduces along the height of the frame. The rebars are mainly placed along the short sides of the cross-section. The experimental investigation conducted in 1999 includes uniaxial compression tests on concrete and uniaxial tensile tests on steel rebars. The compressive strength of concrete was determined on 11 cylinder samples extracted from the structure. The minimum and maximum values are discarded and the collection of data thus obtained provides an average value of 25.2 MPa. The elastic modulus of concrete is equal to 22000 MPa. The yield strength of rebars was determined for 4 specimens and the average value is equal to 374 MPa.

Type of load	g_k (kN/m^2)	q_k (kN/m^2)
Standard floor	5.10	3.00
Top floor	4.10	1.00
Pitched roof	4.22	1.75
Infill	3.00	--

Table 1: Loads per square meter.

3 NONLINEAR NUMERICAL MODELS

3.1 Common features of the numerical models

Two three-dimensional numerical models are built by each research unit to predict the seismic response of the case study building. The difference is in the treatment of the masonry infills, whose contribution to the lateral stiffness and strength of the structure is neglected in the first numerical model (bare frame model) while it is considered in the second one (infilled frame model). Beams and columns are modelled by one-dimensional beam-column elements, while infills (if modelled) are simulated by a pair of equivalent diagonal trusses. Figure 2 shows a schematic view of the structural model. The columns of the first storey are clamped at the base. The in-plan position of each column is coincident with the centroid of its cross-section at the first storey. The model describes the four floor decks and the pitched roof. The part of the deck sustained by the transversal frame located in the right side and by the Gerber joint is not explicitly modelled; in particular, it is included in the numerical model only considering its contribution in terms of gravity load and mass. Since the floor decks are endowed with a concrete slab, their in-plan stiffness is assumed very large, even though this is achieved by means of different modelling strategies by the research units. The mass of each floor is determined from the gravity loads in the seismic design combination. The mass of the pitched roof was added to that of the fourth floor. The floor masses, which are resumed in Figure 2, are distributed among the nodes of the floor on the basis of their tributary areas. The mass of the part of the deck not explicitly modelled is added to the mass of the right side nodes.

Gravity loads are introduced into the numerical model in the form of loads distributed on the beams and forces applied in the nodes of the upper ends of the columns. The nodes of the right side of the deck are loaded also with the forces transmitted by the part of the deck that has not been explicitly modelled.

Floor masses

Floor	Mass (t)
4th	441.4
3˚	354.8
2nd	345.4
1st	347.6

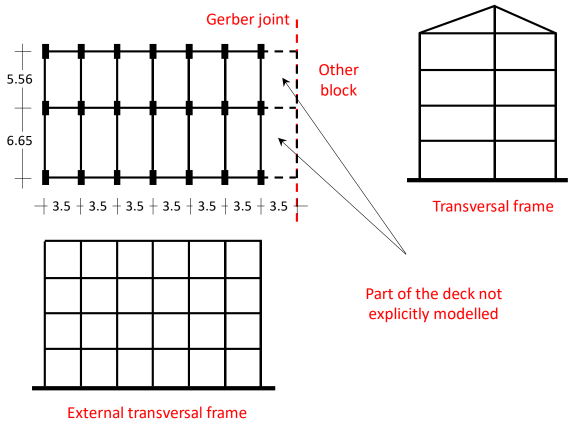

Figure 2. Schematic view of the numerical model and floor masses.

The same material properties of concrete and rebars are assumed for the development of all the numerical models. Compression strength f_c and elastic modulus E_c of concrete are assumed equal to the value determined in Section 2, i.e. 25.2 MPa and 22000 MPa, respectively. The strain corresponding to the peak compressive stress is equal to $2x10^{-3}$. The tension strength of the concrete is neglected. The yield strength f_y of the rebars is assumed equal to the value determined in Section 2 (374 MPa) and the elastic modulus is equal to 200000 MPa.

The columns of the perimeter longitudinal frames are modelled by two finite elements joined in an intermediate node. In the infilled model, these nodes allow the insertion of the equivalent trusses used to simulate the partial height infills. No tension strength is assigned to the equivalent trusses, while their response in compression is simulated by the trilinear force-displacement relationship based on the proposal of Decanini et al. [1-3]. Openings in the infills are taken into account reducing lateral stiffness and peak strength according the suggestions of Decanini et al. [2]. The parameters that control the model are set based on the data collected by Liberatore et al. [4] and elaborated within the RINTC project [5].

3.2 Model of UniRM3 with distributed plasticity finite elements

The numerical model of the research unit UniRM3 (University of Roma Tre) is developed in OpesSEES [6] environment and simulates beams and columns by finite elements with distributed plasticity based on the iterative force-based formulation (*forceBeamColumn* element). The beams are simulated with three or one element depending on the variation of the rebar configuration along the longitudinal axis. The columns are simulated by one or two elements depending on the presence of the infills. The number of integration points is three or five depending on the length of the finite element. The concrete part of the cross-sections is discretized in fibres with area of about 2x2 cm^2. The longitudinal rebars are simulated by single fibres placed on the cross-section in their actual position. The uniaxial model *Concrete04* without tensile resistance is adopted to simulate the stress-strain relationship of concrete according to Mander et al. [7], while the response of the steel rebars is simulated by means of the Giuffrè-Menegotto-Pinto model [8] and implemented in OpenSEES as *Steel02*. The isotropic hardening is neglected, while the parameter b responsible for the kinematic hardening is assumed equal to 0.005. Finally, the *Hysteretic material* model is adopted to simulate the infill behaviour. Each material parameter is calibrated on the basis of the material characteristics defined in Section 3.1 and the same concrete properties are assumed for the confined and unconfined parts in accordance with the models of the others research units. The *rigidDiaphragm* command of OpenSEES is used to create in-plan rigid constraints between the nodes of the floors.

3.3 Models with fibre discretization and plastic hinges of finite length

The numerical models developed by the research units of UniCT (University of Catania) and UniSA (University of Salerno) simulate beams and columns by finite elements with plastic hinges at their ends based on force formulation. The model is developed in the OpesSEES environment and the *beamWithHinges* element is used. The length of the plastic hinges of the beams is assumed equal to the depth of the cross-section while the one of the columns is equal to the average of the two dimensions of the cross-section. The central part of the elements is elastic and the elastic modulus is reduced to account for the cracking of the concrete. In particular, it is assumed equal to 50% and 90% of the elastic modulus assigned to the concrete (22000 MPa) for beams and columns, respectively. For all the analysed elements, the cross-section is discretized in 2x2 cm fibres. The longitudinal rebars are simulated by single fibres placed in their actual position. The steel reinforcement of the beams considered in the model is that effectively anchored, as deduced from the technical drawings. The uniaxial models *Concrete04* without tensile resistance, *Steel02* and *Hysteretic material* are adopted to simulate the cyclic response of concrete, rebars and infills (in the infilled model), respectively. The main difference between the numerical models of the two research units is the strategy adopted to simulate the presence of the floor concrete slabs. This is simulated in the model of UniCT by rigid diaphragms that mutually connect the nodes of the floor decks. Beams are connected to the rigid diaphragms by axial buffer elements to avoid fictitious axial forces caused by the interaction with the rigid diaphragm [9]. Instead, the model of UniSA replicates the stiffness of the floors by elastic concrete trusses connecting opposite corners of the floor decks. Elastic modulus of the trusses is equal to 22000 MPa and their cross-section is assumed to be equal to 4 cm deep and 70 cm wide (about 1/10 of the truss length).

3.4 Models with phenomenological lumped plasticity elements

The research units of PoliBA (Politecnico di Bari) and UniNA (University of Naples Federico II) reproduce beams and columns by using a phenomenological lumped plasticity approach. In particular, elastic elements with nonlinear rotational springs (plastic hinges) at the two ends are used. Plastic hinges are located at the critical zones: end cross-sections of beams and columns for the bare frame model and additionally, for the model of the infilled frame, at the end cross-sections of the captive columns. The computer codes used to run the models of PoliBA and UniNA are SAP 2000 [10] and OpenSEES, respectively. Preliminarily, a sectional analysis is performed to define the backbone of the moment-rotation M-θ. In the case of columns, the sectional analysis is performed assuming an axial load value equal to that due to gravity loads. Internal diaphragm constraints are assigned to each floor, consistently with the assumed condition of rigid floor.

The research unit of PoliBA assumes a moment-rotation relationship with plastic hardening and post-peak softening. The yielding moment M_y, the ultimate moment M_u, the yield rotation θ_y and the ultimate rotation θ_u are determined by the sectional analysis and the formulas suggested by the Italian Seismic Code [11]. The sectional analysis is performed considering the constitutive laws stipulated in the Italian Building Code: parabola-rectangle for concrete and elastic-plastic for steel. The residual moment resistance is assumed equal to 0.2 M_y. No reduction is applied to the concrete stiffness.

The moment-rotation relationship of the research unit of UniNA reproduces the first cracking, yielding, maximum, ultimate (20% strength drop) and zero resistance point of the cross-section. First cracking and yielding moments are calculated by a sectional analysis. The maximum moment and the chord rotation values defining the response backbone are calculated by the empirical expressions proposed by Verderame and Ricci [12] and calibrated for RC elements with plain bars.

3.5 Models with phenomenological beams and fibre discretization of columns

The research units of UniCH (University of Chieti-Pescara) and UniAQ (University of L'Aquila) adopted a phenomenological lumped-plasticity approach for the modelling of beams, and finite elements with distributed plasticity and fibre discretization of the cross-sections to simulate the columns. Internal diaphragm constraints have been assigned to the nodes of each floor to simulate the presence of the concrete slab. The numerical models of the research units of UniCH and UniAQ are developed in OpenSEES and SAP 2000 environment, respectively.

The research unit of UniCH modelled the beams of the analysed structure by elastic members with concentrated plastic hinges at their ends. The plastic hinges are simulated by nonlinear zero-length elements whose moment-rotation relationship is reproduced by the phenomenological model of Ibarra et al. [13]. In particular, the *ModIMKPeakOriented* uniaxial material of OpenSEES, which adopts a deterioration model with peak-oriented hysteretic response [14], is used. The parameters that control the model are determined by the equations proposed by Haselton et al. [15]. The *forceBeamColumn* element of OpenSEES with three integration points is used for each column segment. The uniaxial models *Concrete04* without tensile resistance and *Steel02* are used for concrete and rebars, respectively. The isotropic hardening is neglected, while the parameter b responsible for the kinematic hardening is assumed equal to 0.005. The uniaxial material *Concrete01* is used to replicate the cyclic response of the infills in the infilled model.

The zero-length plastic hinges of the beams of the model developed by the research unit of UniAQ are characterized by a moment-rotation law defined in accordance with FEMA356 [16]. Instead, the columns are modelled by means of elastic beam-column elements provided by fibre plastic hinges of finite length at the two ends. In particular, the cross-section of the plastic hinges is discretized adopting a mesh of 15x15 fibres. The length of the plastic hinges is set equal to the average size of the columns cross-sections along the two directions. Neither for beams, nor for columns the stiffness reduction due to concrete cracking is considered. Concerning the infilled numerical model, *non-linear link* elements characterized by "multilinear plastic" properties in accordance to Decanini law have been introduced to model the masonry infills.

3.6 Numerical model displacement-based finite elements

The research unit of UniPR (University of Parma) developed a numerical model using Abaqus 2018 software [17]. Displacement-based finite elements B31 and B32 are adopted depending on the number of elements used for the beam and column discretisation and depending on assumed plastic hinge length. For beams, the nonlinear response is considered only for bending moment about the local 1-axis of the cross-section and is defined specifying the moment-curvature relationship M_1-χ_1. For columns, the nonlinearity is assigned to the responses to the bending moments about the local axes 1- and 2-. It is assumed that these nonlinear responses are uncoupled. The moment-curvature relationships are obtained by means of the Biaxial software considering a parabola-rectangle stress-strain relationship for concrete (compressive strength equal to 25.2 MPa, peak and ultimate strains equal to $2\text{x}10^{-3}$ and $3.5\text{x}10^{-3}$) and a plastic hardening behaviour for rebars (yield strength equal to 374 MPa, elastic modulus equal to 200000 MPa, post-yield modulus equal to 500 MPa, and ultimate strain equal to $4\text{x}10^{-3}$). The M_1-χ_1 and M_2-χ_2 relationships assigned to the columns are determined considering the axial force caused by gravity loads. The moment-curvature relationship is enriched with a linear descending part that connects the peak point of the curve to a zero moment in correspondence of a curvature equal to 4.5 time the peak curvature value. The effect of the floor concrete slab is simulated connecting the floor nodes by diagonal rigid trusses.

4 NONLINEAR ANALYSIS AND RESULTS

The numerical models are analysed by pushover analysis. The gravity loads in the seismic design combination has been applied in load step 1. Hence, horizontal forces are applied alternatively in the longitudinal (*x*-) and transverse (*y*-) direction. The distribution of applied forces is proportional to the floor masses. The results of the analyses are used to predict both local and global response, and the collapse mechanism of the building.

4.1 Analysis of the local response and collapse mechanism

The local response of beams and columns to the incremental loading is represented in terms of moment-rotation relationship. In the case of members modelled by finite elements with lumped plasticity, the plastic rotation is plotted against the bending moment. Instead, when distributed plasticity models are used, the curvature of the end cross-section is obtained from the output returned by computer code. Hence, the rotation is determined multiplying the curvature by the assumed length of plastic hinge. The response of the end cross-sections of all the members is analysed to detect where the yielding localises and identify the collapse mechanism of the structure.

Figure 3 shows the results of the middle column of the interior frame arranged along the *x*-direction and the beams framing into this column. The column is marked with a red circle in Figure 3. The results plotted herein are obtained by means of the pushover analysis of the bare frame models subjected to forces in *x*-direction. The moment-rotation relationship is plotted only for the cross-sections that have yielded under the incremental loading. All the numerical models predict the formation of the plastic hinges in the same cross-sections, i.e. the bottom cross-section of the fist storey column and the end cross-sections of the beams of the four floors. All the models, with the exception of those developed by the research group of UniRM3 and UniSA, predict the same peak moment of the beam. Instead, the beams of the numerical model of UniRM3 exhibit much larger bending moment due to the effect of the compressive axial force induced in the beams by the rigid diaphragm [9]. This effect is mitigated in the model of UniSA modelling the floor concrete slab by deformable trusses rather than a rigid constraint. Nevertheless, the peak moments returned by this model are generally larger than those returned by the models with lumped plasticity beams and by the model of UniCT, which adopts buffer elements on the beams to eliminate the fictitious axial force. The considered numerical models provide similar results also for the columns. Indeed, all the numerical models basically provide the same peak moment response of columns, with differences only in the post-peak phase. In fact, finite elements with distributed plasticity and fibre discretization of the cross-section (UniAQ, UniCH, UniCT, UniRM3 and UniSA) have exhibited strong degradation of the moment resistance. The resistance degradation is less significant in the finite element of UniPR or is even missing in the elements of PoliBA and UniNA. Note that the results illustrated in Figure 3 are qualitatively the same for the other columns and beams. Hence the considerations can be extended to the whole structure and the yield pattern illustrated in Figure 3 for one column reflects the collapse mechanism of the bare framed structure.

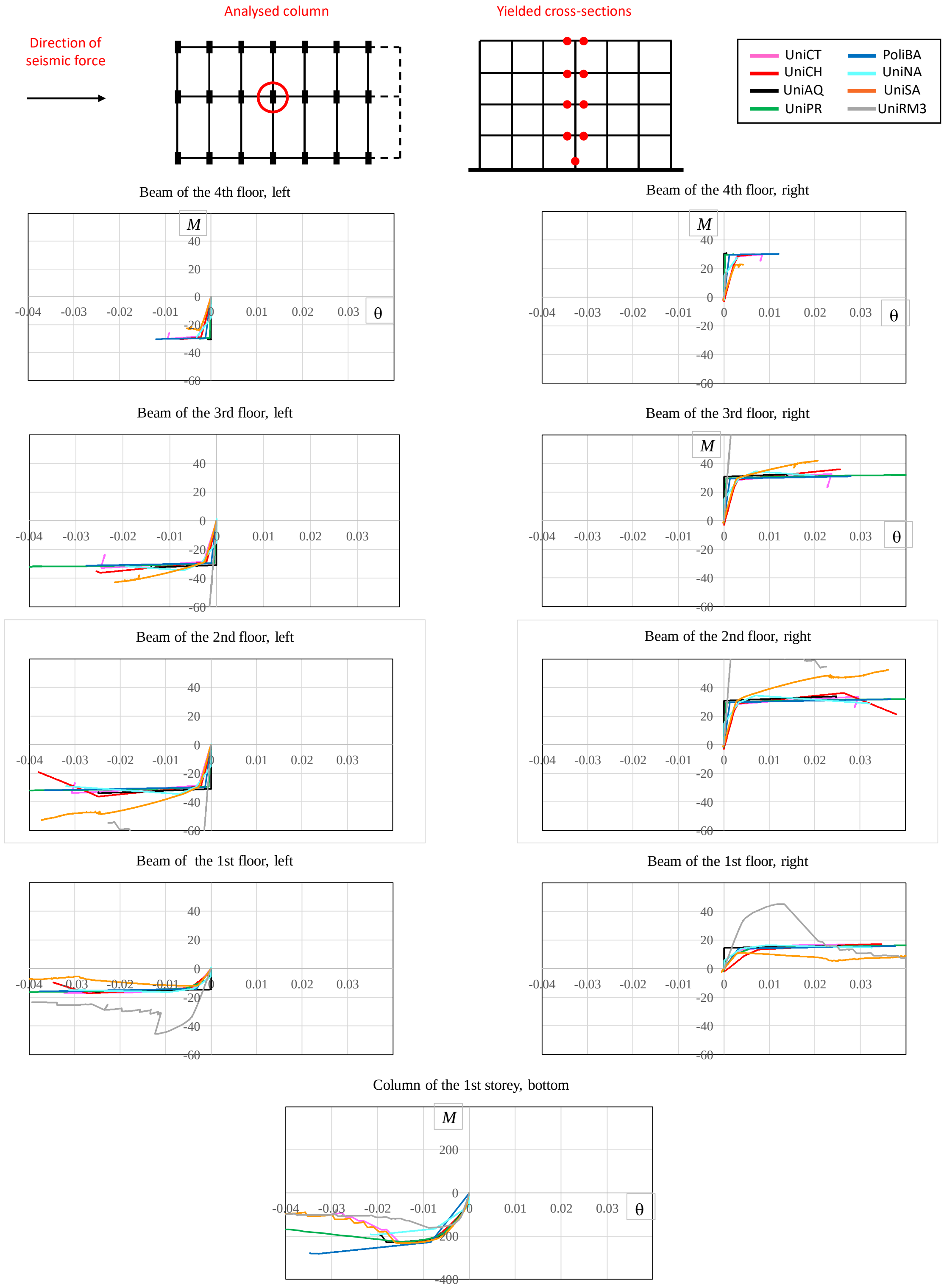

Figure 3. Collapse mechanism of the bare frame models under forces in *x*-direction and moment-rotation relationships of the yielded cross-sections.

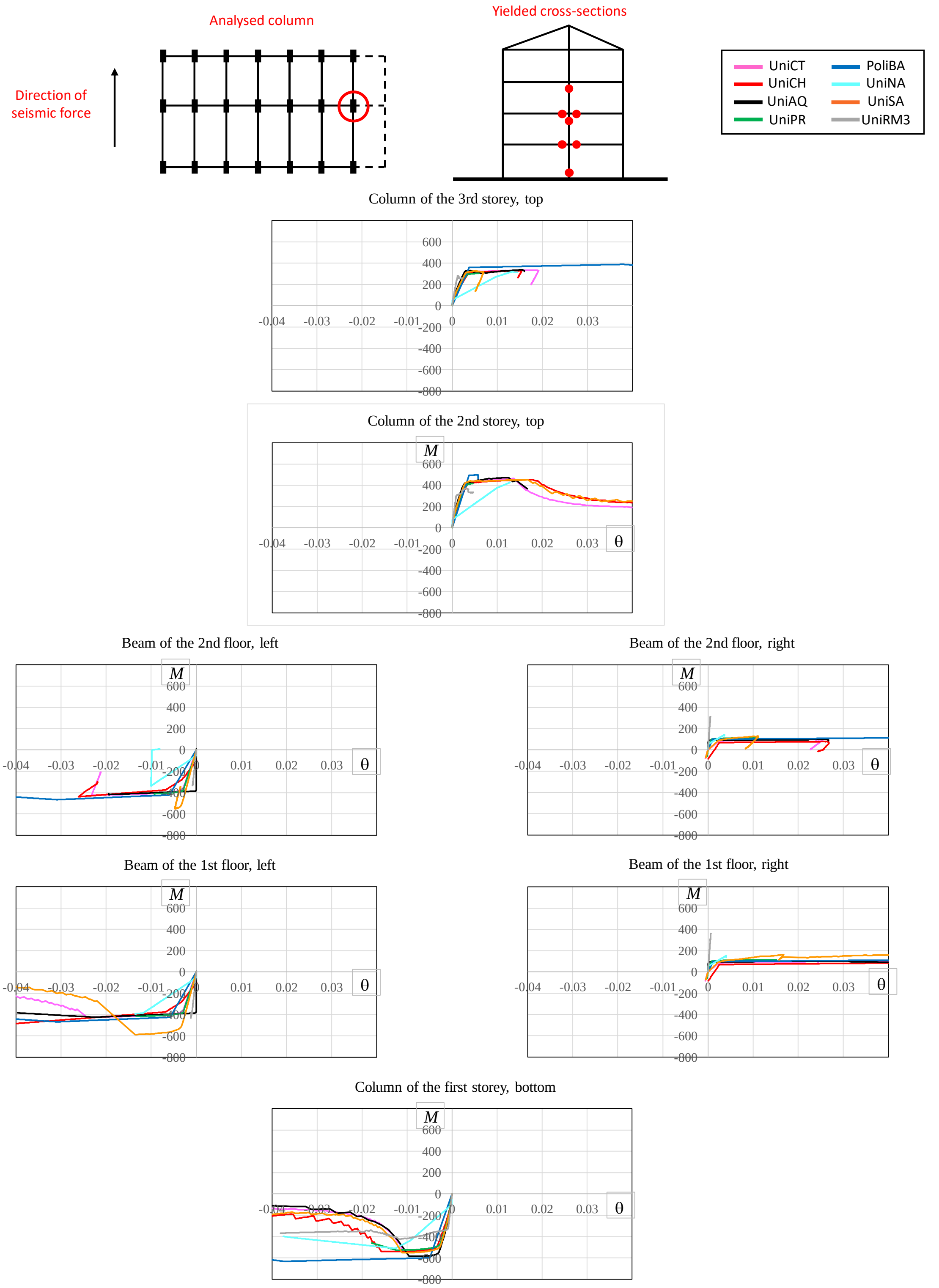

Figure 4. Collapse mechanism of the bare frame models under forces in *y*-direction and moment-rotation relationships of the yielded cross-sections.

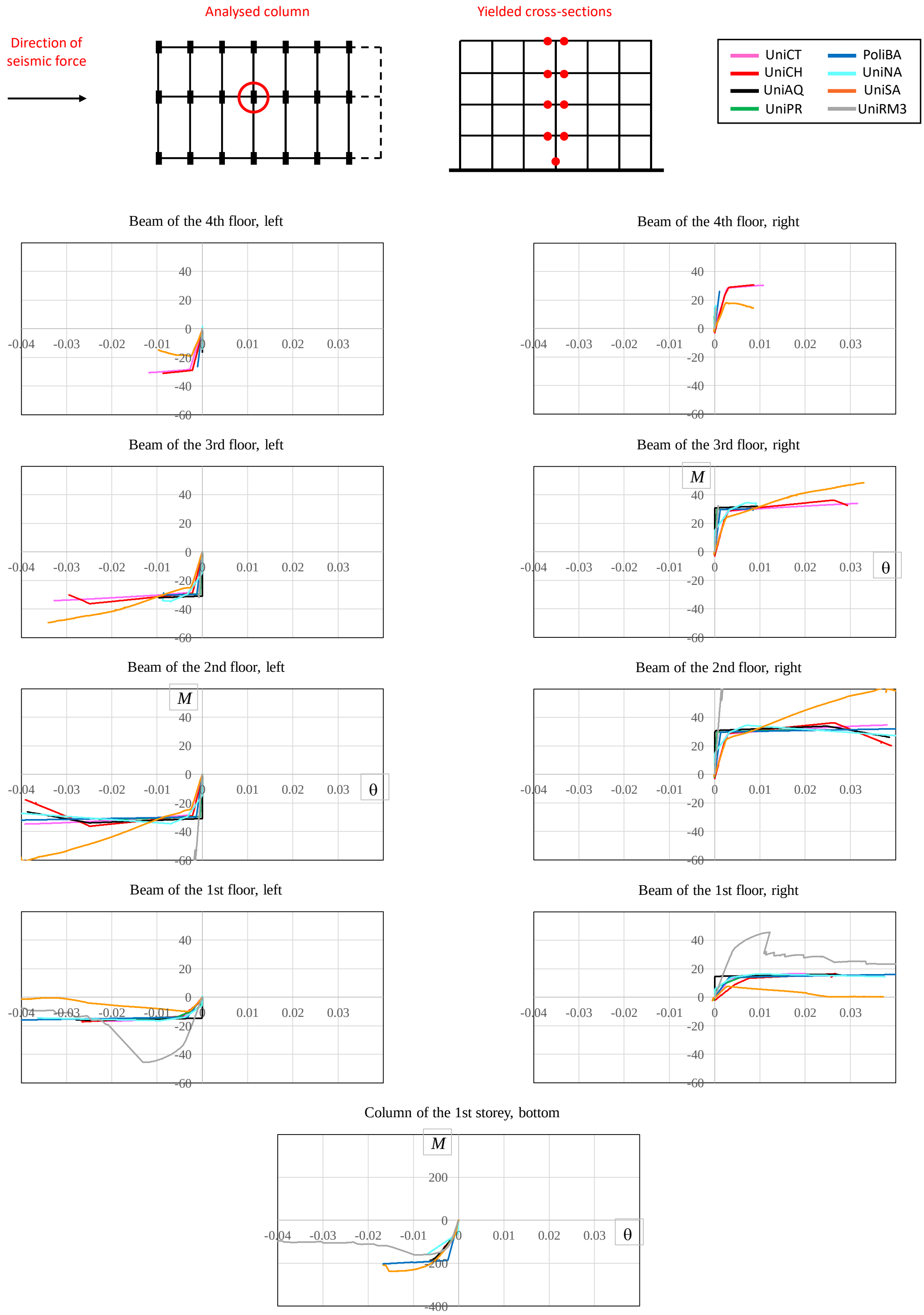

Figure 5. Collapse mechanism of the infilled frame models under forces in *x*-direction and moment-rotation relationships of the yielded cross-sections.

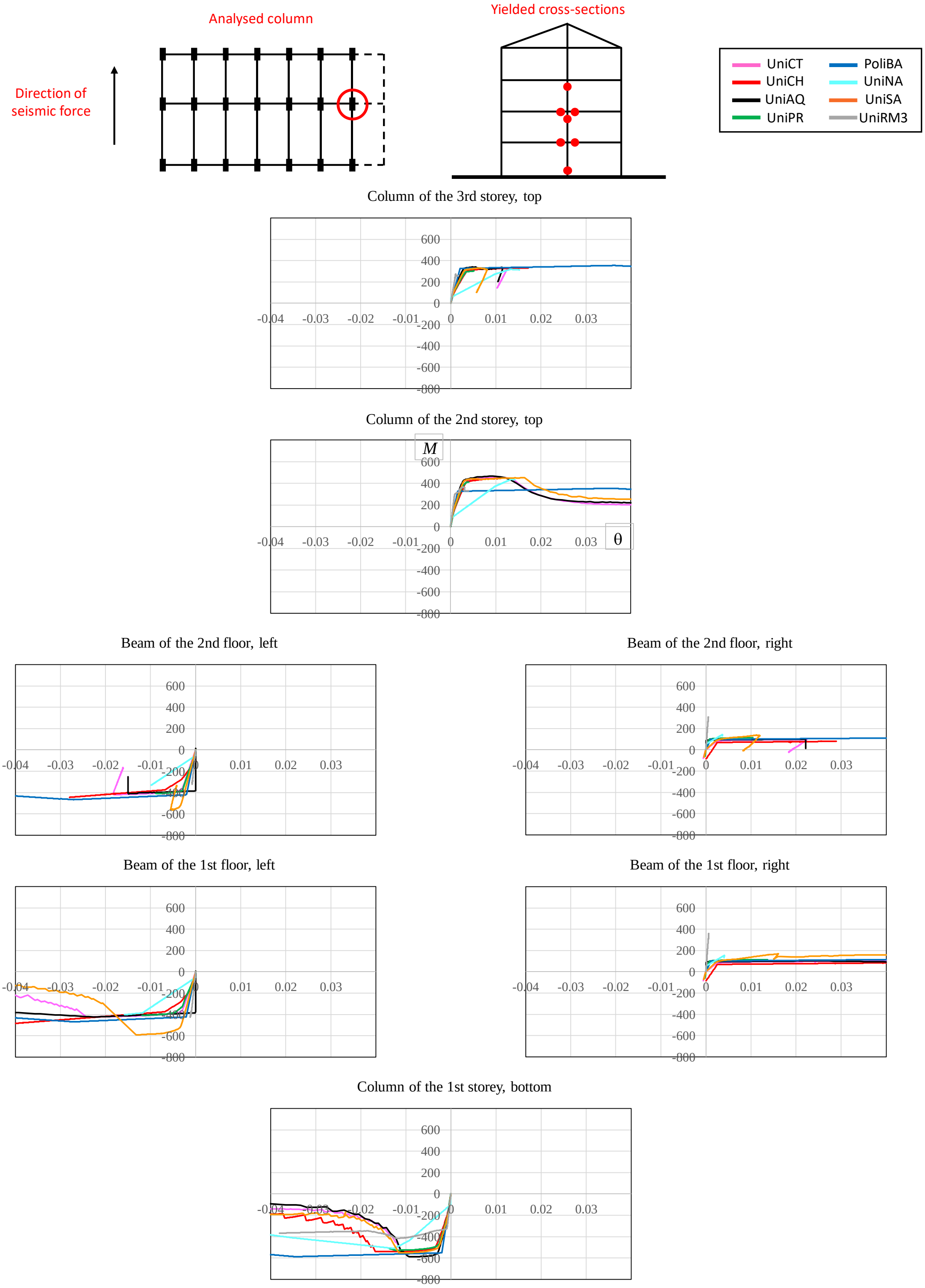

Figure 6. Collapse mechanism of the infilled frame models under forces in *y*-direction and moment-rotation relationships of the yielded cross-sections.

Figure 4 reports the results of the bare frame structure loaded with forces in *y*-direction. The results refer to the middle column of the frame arranged along the *y*-direction and located on the right side of the building. This frame has been selected because it is the one that sustains the largest displacement demand. Indeed, the mass is not symmetrically distributed with respect to the rigidity centre of the structure and the centre of mass is located at its right. The moment resistance of the beams of the frames arranged along the *y*-direction is comparable to that of the columns. Consequently, all the numerical models detect a two-storey collapse mechanism; the beams yield at first and second and floor, while remain in the elastic range of behaviour at third and fourth floors, and the column yields at the bottom cross-section of the first storey and at other location as well. The considerations on the agreement/disagreement between the local responses predicted by the considered numerical models apply also for this analysis.

Finally, Figures 5 and 6 summarize local response of members and collapse mechanism of the infilled frame numerical models. Regardless of the direction of loading, the yielding pattern of the analysed columns is the same as that found in the case of the bare frame structure. Furthermore, the relation between the local responses predicted by the considered numerical models replicate those found in the case of the bare frame models.

4.2 Analysis of the global response

The global response of the structure is represented in Figure 7 in terms of base shear – roof displacement relationship. The results evidence that the structure is significantly stiffer and stronger when loaded in *y*-direction. When the structure is loaded in *x*-direction, the bare frame models with lumped plasticity elements (PoliBa, UniAQ, UniNA and UniPR) are less flexible than those with finite element with distributed plasticity or plastic hinges of finite length and fibre discretization of the cross-section (UniCH, UniCT, UniRM3 and UniSA). The good agreement between the lateral stiffness of the latter models is the result of a proper reduction of the elastic modulus of the elastic segments of the members modelled with plastic hinges of finite length. These observations are confirmed also in the case of the infilled frame models. Instead, in the case of the analysis executed with forces in *y*-direction, the initial lateral stiffness is similar for all the models. The only exception is the infilled frame model of PoliBA, whose lateral stiffness is significantly larger than that of its counterparts.

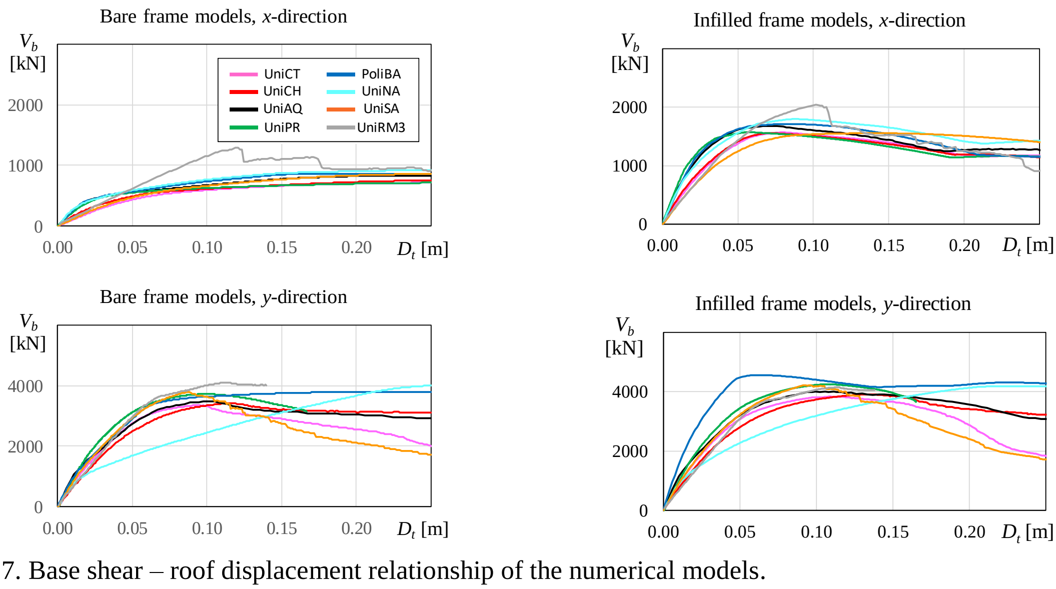

Figure 7. Base shear – roof displacement relationship of the numerical models.

The considered numerical models generally provide similar lateral resistance. The numerical model of UniRM3 exhibits a significantly larger lateral resistance, which reflects the differences already observed in the local responses analysed in Section 4.1. If this case is excluded, the maximum percentage difference is recorded in the case of pushover analysis with forces in *x*-direction of the bare frame models; the minimum lateral resistance, obtained by the model of UniCT, is 25% smaller than that determined by the model of UniPR.

A good agreement between the numerical models is generally achieved on the prediction of the roof displacement corresponding to the peak lateral strength. The only two exceptions are the model of UniNA, whose displacement is much larger than those obtained by the other models for pushover analysis with forces in *y*-direction, and the infilled frame model of PoliBA loaded in *y*-direction, that conversely exhibits the smallest displacement corresponding to peak resistance.

The lateral response evaluated by the pushover analysis with forces in *y*-direction is characterised for some models by a significant strength degradation. Indeed, the strength degradation in the post-peak response is the most important difference between the responses predicted by the considered numerical models. It ranges from the virtually null value exhibited by the models of PoliBA and UniNA to the 60% reduction of lateral strength recorded for the model of UniSA at the roof displacement of 250 mm.

5 CONCLUSIONS

The paper presents the results of a cooperative research project devoted to compare different nonlinear modelling techniques for the prediction of seismic response of buildings. A real building, one of the blocks of the De Gasperi-Battaglia school building in Norcia, is used as case study and is analysed by pushover analysis in two directions. Eight research units shared the data on geometrical, dynamic and mechanical features of the building and developed their own numerical model. Each research unit carried out the analysis two times, including and not including the infills in the numerical model.

A good agreement between the results obtained by the considered numerical models is generally observed in the four cases analysed: pushover analysis in *x*- and *y*-direction including and not including infills. All the models have basically predicted similar moment response of the members and have detected the same collapse mechanism. With few exceptions, the considered models return similar base shear – roof displacement relationship until the attainment of the peak lateral resistance. Instead, in the post-peak phase the response in terms of base shear may be quite scattered. This is evident when the building is pushed in *y*-direction. In this case, the lateral strength of some models exhibits significant degradation while it remains virtually constant and close to the peak value for others.

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