Nonlinear modeling approaches for existing reinforced concrete buildings: the case study of De Gasperi-Battaglia school building in Norcia

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Nonlinear modeling approaches for existing reinforced concrete buildings: the case study of De Gasperi-Battaglia school building in Norcia

Tecniche di modellazione non-lineare di edifici a telaio esistenti in c.a.: il caso-studio dell'edificio scolastico De Gasperi-Battaglia di Norcia

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ABSTRACT: This paper reports on a collaborative activity developed as part of the DPC-ReLUIS Research Project, year 2017. It aims at comparing the results obtained by considering alternative options in the definition of the nonlinear FEM model employed in pushover analyses for seismic assessment of existing RC frames. Specifically, the RC frame structure of the De Gasperi-Battaglia school building located in Norcia, Italy, is considered as a relevant case-study. This structure has been designed in the '60s of the past century according to the seismic code of the time and, hence, without taking into account the principles of Capacity Design. Although the building was actually retrofitted before the 2016 Central Italian earthquake, in this paper its original configuration has been considered. The nonlinear behaviour of the frame structure has been modelled by both following alternative approaches and employing different analysis codes. Therefore, this paper proposes an overview about how different the simulation output can be as a result of different modelling and analysis choices. In doing that, the work can be relevant to practitioners, as they may be warned about the consequences of those choices in terms of seismic vulnerability evaluation. / Il presente lavoro illustra l'attività di collaborazione sviluppata all'interno del progetto di ricerca DPC-ReLUIS per l'anno 2017. Esso ha lo scopo di comparare i risultati ottenuti a partire da diverse ipotesi adottate per la definizione di un modello FEM non lineare utilizzato nell'analisi PushOver per la verifica sismica di strutture esistenti. In particolare, è stato analizzato il caso studio dell'Istituto scolastico De Gasperi-Battaglia sito a Norcia, realizzato negli anni '60 del secolo scorso secondo i codici normativi dell'epoca e, quindi, senza considerare i principi del Capacity Design. Sebbene tale struttura e stata rinforzata prima del terremoto del 2016 del Centro Italia, in questa memoria essa è studiata nella sua configurazione originale. Il comportamento non lineare della struttura è stato modellato e analizzato sia con differenti approcci, sia con differenti codici di calcolo. In definitiva, il presente lavoro propone una disamina di come può variare l'output delle analisi al variare delle scelte operate all'atto della modellazione e dell'analisi. Nel fare ciò, il lavoro sviluppato vuole essere un utile documento per i professionisti e per i tecnici impegnati nelle procedure di verifica di edifici esistenti in CA, illustrando le conseguenze delle scelte adoperate in fase di modellazione.

KEYWORDS: RC existing buildings, non-linear analysis, pushover, Finite Element Method, frame structure / strutture esistenti in calcestruzzo armato, analisi non lineari, pushover, elementi finiti, struttura a telaio

1 INTRODUCTION

Seismic assessment of existing buildings is a challenging task for practitioners, as it is influenced by a number of problematic aspects (Franchin et al., 2010), such as uncertainties about relevant geometric quantities (Silva et al., 2012), limited knowledge of structural detailing (Jalayer et al., 2010) and high variability in material properties (De Stefano et al., 2013) possibly affected by both the original workmanship practice (Tabbakhha & Modaressi- Farahmand-Razavi, 2016) and eventual degradation phenomena (Li, 2004). Moreover, when it comes to Reinforced Concrete (RC) structures, specific concerns arise with respect to the actual accuracy and reliability of capacity models, namely those mathematical relationships intended at determining the members' strength and ductility based on the available knowledge about materials properties and structural detailing (fib, 2003). Furthermore, a significant variability in the results of seismic assessment may derive from the adoption of alternative analysis methods chosen among those generally accepted for the seismic simulation of structures (Fragiadakis et al., 2013).

This paper aims at summarizing the work made by several research groups as part of WP2 of the DPC-ReLUIS research project (year 2017) about the comparison of seismic simulation results obtained from alternative modelling approaches about both capacity models and numerical techniques for seismic analysis. Specifically, the results of pushover analyses run by considering either lumped- or distributed-plasticity models are proposed and discussed with the aim to highlights their potential and drawbacks. Since those analyses are performed by using commercial analysis codes, such as OpenSEES (Mazzoni et al., 2010), Abaqus (Dassault Systèmes Simulia Corp., 2017), MidasGen (CSPFEA, 2018) and SAP2000 (Brunetta et al., 2006), the proposed results and comments are intended as a tutorial guide for practitioners in their everyday work.

Section 2 outlines the general theoretical bases of the various modelling approaches targeted in this study. Then, to make things clearer, Section 3 proposes a relevant case study (namely, the De Gasperi - Battaglia school building located in Norcia): the RC structure is described into details in its “as-built” configuration designed and realized in the ‘60s of the past century and, hence, without considering the recently completed seismic upgrading intervention. Section 4 highlights the main lessons learnt from this study and discloses the main ideas about the future developments of the present collaborative research.

2 NONLINEAR MODELLING OF MATERIALS AND ELEMENTS

Seismic analysis of structures, especially those executed on existing ones and aimed at determining their seismic vulnerability, are generally carried out on nonlinear Finite Element (FE) models employed either in static (namely pushover) or dynamic (namely time-history) simulations of the response under actions induced by earthquake shaking.

However, several levels of detailing can be chosen in FE models, depending, on the one hand, on the actual accuracy of the available data (in terms of geometry, material properties and structural detailing) and, on the other hand, on the computational efforts that can be afforded. Ferretti et al. (2002) propose a possible classification of the FE employed in structural and seismic analyses in a decreasing order of accuracy and computational effort:

- general purpose *3D elements* capable of simulating the structural response under the general assumption of continuum mechanics with non-linear constitutive laws: Abaqus (Dassault Systèmes Simulia Corp., 2017) is one of the most widely employed codes featuring these kinds of models (Belletti et al. 2017);
- *fiber beam elements* formulated by assuming either the kinematic (displacement-based elements) or the equilibrium (force-based elements) conditions assumed in common beam theories (e.g. the Timoshenko theory) with non-linear properties implemented by means of 1D stress-strain relationships referred to the *fibers* of specific transverse sections selected throughout the beam axis and assumed as sampling points: this approach, generally referred to as “distributed plasticity approach” is available in various codes, among which the OpenSEES (Mazzoni et al., 2010) and MidasGen (CSPFEA, 2018);
- *sectional beam elements*, similar to the aforementioned fiber-beam models, in which the nonlinear behavior is defined in terms of moment-curvature relationship in specific sampling points through the element’s axis: although they are based on a distributed plasticity approach, relevant aspects of the mechanical behavior (such as the M-N interaction on both moments and curvatures) cannot be generally taken into account (elements of this sort were employed within the code IDARC (Reinhorn et al., 2009), among the first made available to the scientific community for performing nonlinear seismic analyses);
- *plastic-hinge beam elements*, in which the nonlinear behavior is *concentrated* (or *lumped*) in specific sections (e.g. the extreme sections of columns) whose moment-rotation relationship simulate the non-linear structural response: these elements are employed in several FE codes, among which SAP2000 (Brunetta et al., 2006); the N-M interaction can be taken into account in advanced formulations of this class of elements by either defining a family of moment-rotation curves obtained at different values of axial force (SAP2000) or updating “run-time” those curves by means of a fiber-discretization of the plastic-hinge sections (OpenSEES).

2.1 *Concrete*

The mechanical behavior of concrete is characterized by complex non-linearities, both in compression and in tension. Concrete under compressive stress shows an elastic-linear behavior up to about 1/3 of the maximum resistance: non-linear behavior takes place due to the cracking processes that, after the peak in strength, originates softening phenomena. Post peak softening behavior is strongly influenced by both size of the specimen and boundary conditions (fib-MC2010, 2012). The simulation of seismic response of existing buildings is generally referred to the average compressive strength f_{cm} whose relationship with the other materials’ properties is described by well-established laws provided by various documents, such as NTC (2008) and fib-MC2010 (2012). Moreover, transversal confinement influences displacement and force capacity of RC members. General models are available in the literature for simulating the response of concrete subjected to tri-axial stress states (Kupfer et al., 1969). However, due to weak and widely spaced stirrups generally adopted in existing structures, the effect of confinement is often neglected in seismic assessment analysis.

Concrete behavior in tension is even more complex, as tension-induced cracks are discrete in nature, whereas FE models are generally based on assuming the continuity of displacement and force fields' FE This makes inappropriate the adoption of the classical "deformation" assessment in case of cracked concrete, for which the crack localization effect is fundamental in FEM analysis (fib, 2008; Bazant, 1993). An approach based on crack opening is provided by fib-MC2010 (2012). Generally, these techniques are based on the definition of fracture energy G_f, which can be determined through experimental tests or calculated in accordance to specific formulations, such as those provided by fib-MC2010 (2012): G_f=73 $f_{cm}^{0.18}$. Moreover, the concrete average tensile strength f_{ctm} can be calculated as suggested by fib-MC2010 (2012).

Several uniaxial σ-ε laws describing the behavior of concrete are available in the literature. For instance, the work by Kent & Park (1971) was further developed by Scott et al. (1982) considering the cross-section confinement, which represents a very important factor in case of cyclic loads. Popovics (1973) proposed a (σ-ε) relationship similar to Kent and Park's one, in which no hysteretic cycles are considered in the unloading/loading branches. Mander et al. (1984) proposed a model capable of simulating the hysteretic behavior of confined and unconfined concrete under cyclic compression and tension.

2.2 *Steel*

Generally, the mechanical behavior of steel reinforcement can be assumed symmetrical in compression and in tension. In monotonic load conditions, this behavior is characterized by a linear elastic initial branch up to the yielding point, after which a plastic- behavior first and a subsequent hardening phase are later noticed till the failure is reached.

As for the cyclic behavior, in case no sign reversal, loading-unloading curves correspond each other, with almost no hysteresis effects. Therefore, monotone σ-ε curve correspond to the envelope of the cyclic behavior. Conversely, in the case of load reversals, a gradual decrease in yield strength is noticed, together with non-linear phases characterized by a progressive loss in stiffness (Bauschinger effect). Since existing reinforced buildings are characterized by very low amount of transversal reinforcement, constitutive law for longitudinal reinforcement should also take into account buckling phenomenon as well. The monotonic post-buckling behavior varies with slenderness of the longitudinal reinforcement, defined as λ=L/D, in which L represents transversal stirrups spacing and D is the diameter of longitudinal rebars.

Mander et al. (1984), Mau & El-Mabsout (1989) and Monti & Nuti (1992) have studied the influence of the possible buckling of rebars in RC sections. The value λ=5 has been identified as the upper limit of slenderness for the rebar not to exhibit buckling (Zhou et al., 2015). This value is however influenced by the actual mechanical properties of steel (Bae et al., 2005).

Dhakal & Maekawa (2002) demonstrated that compressive behavior of a steel rebar subject to buckling depends on both its yield tension and slenderness, so that different steel grades can generate identical tension-deformation curve if combined parameter $L/D\sqrt{f_y}$ results the be the same. A wide investigation on steel reinforcements used in Italy till the '70s (smooth rebars) are presented by Cosenza & Prota (2006), in which a slenderness-ratio between 5 and 70 have been reported together with a new constitutive model for smooth rebars prone to buckling effects. Prota et al. (2009) reported experimental results highlighting that cyclic behavior of longitudinal rebars characterized for high values of slimness presents marked pinching.

Models available in literature considers the cyclic behavior of steel in a simplified way. The Menegotto-Pinto model (1973), later modified by Filippou et al. (1983), aims to simulate the cyclic behavior of steel and capture the Baushinger effect and the kinematic hardening (Menegotto & Pinto, 1973) and its optionally isotropic nature (Filippou et al. 1983).

2.3 *Modeling of bending and combined compression and bending mechanisms*

As already mentioned at the beginning of Section 2, alternative approaches can be followed with the aim to simulate the nonlinear response of members and structures under seismic actions. For instance, an empirical lumped plasticity macro-modelling approach (referred to as "UNINA-Verderame" in the following) can be used in addition to fiber-based approaches. Nonlinear moment-chord rotation springs are adopted at the end of beam/column (elastic) elements; the backbone of these moment-chord rotation relationships simulate the key points of the nonlinear response (cracking, yielding, maximum, "ultimate" (20% strength drop) and zero resistance). They are defined through the empirical expressions proposed by Verderame & Ricci (2017) calibrated for RC members with smooth rebars and calculated assuming the axial load value due to gravity loads and shear span equal to half the clear length of the element.

3 CASE STUDY

3.1 *General description*

The building studied in the present work is the De Gasperi-Battaglia school Institute located in Norcia (Italy). It is a four-storey RC framed structure with a rectangular shape in plan (12.8 x 59.8 m) and a maximum height of about 16.10 m measured from the foundation level. The entire building consists of three blocks, divided among them by two technical gaps arranged in parallel to the shorter direction. The floor slabs are made in RC with lightweight clay blocks and are arranged in order to transmit vertical loads to transversal frames. Figure 1 depicts the front view of the building in which the two technical gaps are highlighted, while Figure 2 shows the structural scheme in plan (Figure 2,a), section (Figure 2,b) and the tridimensional model (Figure 2,c) of the left block as, in this work, it is analyzed only.

The technical gap that separate the structure under investigation from the central building is a Gerber system transmitting vertical loads of half a bay on the right of the frame T7 to the central block, whereas the total amount of the seismic mass is supported by columns of the investigated building.

The geometric and mechanical properties of the structure (i.e. sections of beam and columns, amount of reinforcement, concrete compressive strength, yielding stress of steel, and so on) results from the *"Seismic Identification Campaign"* performed in 1999 and the design documents of the *"Retrofitting intervention"* developed in 2003 and 2010. Further details are herein omitted for sake of brevity.

3.1 *Description of numerical models*

Numerical models are developed by the Research Units (UR) involved within the DPC-ReLUIS Project without considering the beams of the last bay connecting the analyzed structure with the central building. Therefore, the related loads are applied as concentrated forces on the beam-to-column nodes of the transversal frame labelled T7 (Figure 2,a). As far as the foundations are concerned, the hypothesis of fixed constrain at the base is assumed. Moreover, at this stage of the activities, the mechanical contribution of masonry infills is neglected in the structural model and brittle mechanisms (shear failure of beams, columns and joints) are not considered.

The mass of the structure is evaluated according to the seismic combination (eq. 2.5.5 of NTC, 2008) assigning partial coefficients equal to 0.6 for variable school loads and 1 for gravitational loads. Floor masses result equal to 313.33 ton, 312.27 ton, 309.45 ton and 439.14 ton, respectively at the 1^{st}, 2^{nd}, 3^{rd} and 4^{th} floor. Pushover analyses, which results are reported in section 3.3, are performed by applying a horizontal force distribution proportional to the floor masses. Table 1 summarizes the nonlinear characteristics of the models and the seismic analysis code adopted by each UR.

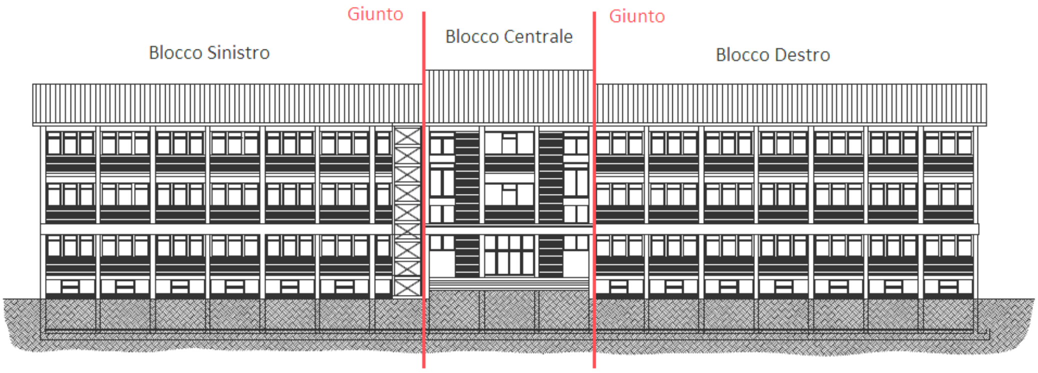

Figure 1. Front view of the structure with the identification of the 3 blocks / Prospetto anteriore dell'edificio con l'individuazione dei tre blocchi

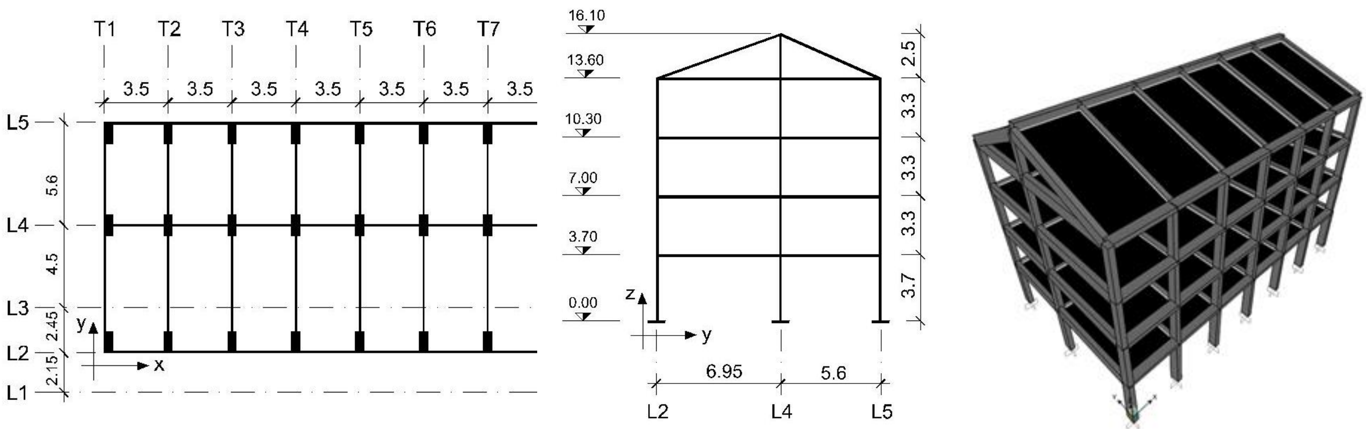

Figure 2. Structural scheme of the analyzed structure: plan view (a), section (b) and tridimensional model (c) / Schema strutturale del corpo di fabbrica analizzato: pianta (a), sezione (b) e modello tridimensionale (c)

Table 1. Modeling features used by UR (NL=type of plasticity, ST=Section-type, C=Concentrated, D=Distributed, Mθ=Moment-curvature law, F=Fiber section) / Caratteristiche di modellazione utilizzate dalle diverse UR (NL=tipo di plasticità, ST=tipo di sezione, C=Concentrata, D=Distribuita, Mθ=legge momento-curvatura, F=sezione a fibre)

UR	Software	Beam		Column	
		NL	ST	NL	ST
UnivAQ	SAP2000	C	Mθ	C	Mθ
UniPR	Abaqus	C	Mθ	C	Mθ
UniCH	MidasGen	C	Mθ	C	Mθ
	MidasGen	C	Mθ	D	F
	OpenSEES	D	F	D	F
UniSA	OpenSEES	C	F	C	F
UniNA Verderame	OpenSEES	C	Mθ	C	Mθ
UniNA Rosati	OpenSEES	D	F	D	F
UniRM3	OpenSEES	D	F	D	F
UniCT	OpenSEES	C	F	C	F
PoliBA	SAP2000	C	Mθ	C	Mθ

UnivAQ and PoliBA work with a concentrate plasticity model (plastic hinges) in SAP2000 software. However, UnivAQ analyses are performed considering FEMA356 (2000) moment-rotation law for plastic hinges located at both ends of beams and columns. The floors are simulated as rigid diaphragms. Conversely, the PoliBA model accounts for M-θ laws adopted by NTC (2008) and each floor in the model is simulated by means of shell elements with thickness equal to the one of the RC slab. UniCH takes into account three different models: two of them are developed in MidasGen software considering concentrated (plastic hinges according FEMA356) or distributed plasticity (fiber elements), while the other one is developed in OpenSEES using fiber elements in both beams and columns. Rigid diaphragms are included in all models for simulating the presence of RC floors.

UniRM3 and UniNA-Rosati analyze in OpenSEES a model with distributed plasticity elements similar to the one used by UniCH. In particular, analyses carried out by UniNA-Rosati use specifically-implemented algorithms for the computation of generalized stress integrals and for the time history analysis.

UniCT and UniSA develop very similar models in OpenSEES adopting concentrated plasticity elements. The non-linearity is concentrated at both ends of beams and columns in which the plastic hinges is modelled with fiber elements (namely “beamWithHinges” element in OpenSEES), while the central part of the element is elastic. The plastic hinges are of finite length equal to the depth of the cross-section for beams and to the average of the two dimensions of the cross-sections for columns. The Young’s modulus of the elastic part of the element is assumed equal to 0.5 and 0.8 times the elastic modulus of concrete. This assumption is intended to take into account the effect of cracking. Different approaches are used in order to simulate the floors: UniCT develops a model with rigid diaphragms and axial load releases for beams (Barbagallo et al., 2018), while UniSA adopt equivalent elastic trusses.

Models with rigid floor diaphragms are also developed and the effects of this a different simulation are discussed in the following section.

UniPR use the software Abaqus and adopt M-curvature laws at the integration point of beam elements. Finally, the model developed by UniNA-Verderame in OpenSEES accounts for a novel concentrated plasticity approach proposed by Verderame & Ricci (2017) to which readers can refer for further details.

3.2 *Analysis of results*

The comparisons of the results obtained by different UR in terms of capacity curves are reported in Figure 3 and Figure 4 for the analyses performed along the Y- and X-direction, respectively.

Considering the two analyzed directions, significant differences can be observed in terms of both maximum strength and stiffness.

Seismic resistant frames are disposed along the Y-direction resulting in higher stiffness and strength than X-direction along which only thick beams are present. Moreover, column cross-sections are all disposed with the strong axis along the Y-direction.

A substantially equal response in term of stiffness and strength is observed considering the results outlined by the various UR. As expected the initial stiffness is quite equal along both the Y- (Figure 3) and X-direction (Figure 4) in most of the case, while relevant differences are obtained in terms of maximum strength especially along the Y-direction (Figure 3). The initial stiffness of the numerical models developed by UniCT and UniSA is significantly smaller than that obtained by the other UR. The difference is evident especially when the building is pushed in X-direction. This is caused by the reduced value of the Young’s modulus assigned to the elastic part of the beamWithHinges elements that correspond to consider the structure cracked from the onset of the loading process.

Appreciable differences strength and peak load displacement are observed for distributed plasticity models accounting only for members’ flexural response. The differences are mainly determined by the different techniques used by the UR to mitigate/eliminate the effect of the “fictitious compression” of beams described in section 3.1. Out of these models, the one developed by UniCT provides zero axial force in the beams and leads to the lowest lateral strength.

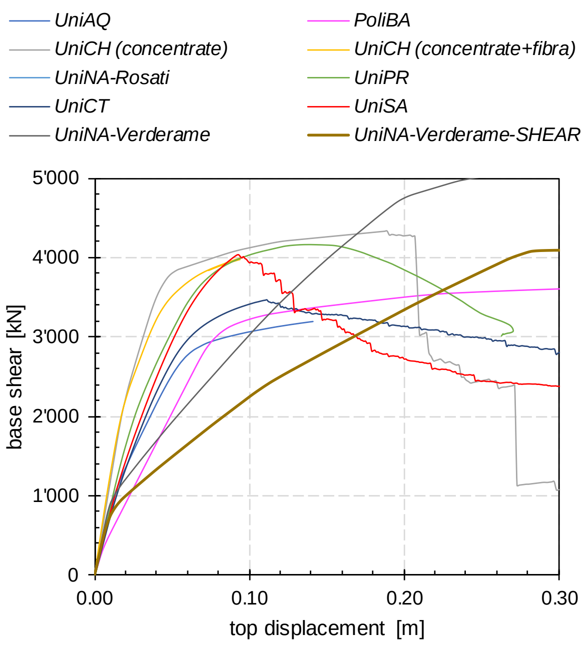

Figure 3. Capacity curves along Y-direction / Curve di capacità in direzione Y.

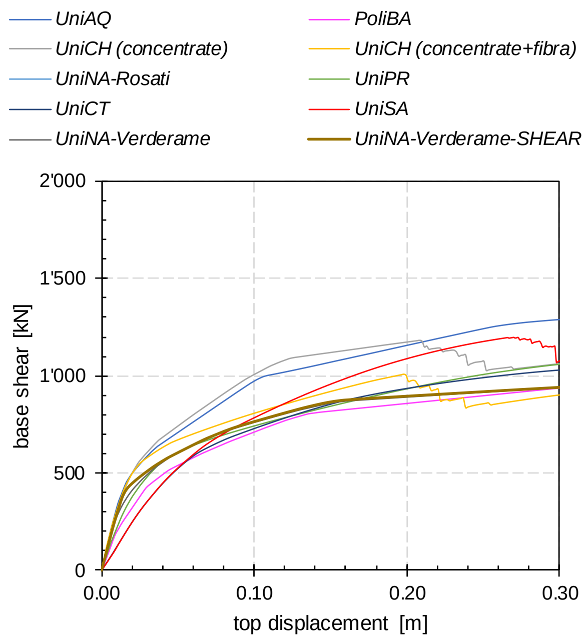

Figure 4. Capacity curves along the X-direction / Curve di capacità in direzione X.

With respect to the distributed plasticity models, different strength and peak load displacement are registered when adopting a lumped plasticity model based on empirical formulations dedicated to RC elements with plain bars (Verderame & Ricci, 2017).

As a matter of fact, apart from the inherent differences due to the adoption of a lumped plasticity approach, with concentrated non-linear moment-chord rotation springs characterized assuming fixed values of shear span and axial load, instead of distributed plasticity elements with fiber sections, the adopted model predicts a peak strength starting from a moment at first yielding calculated a priori through a fiber-section analysis and applying an empirical value of flexural overstrength, observed in the database collected in (Verderame & Ricci, 2017), different from the remaining modeling approaches; furthermore, the empirical calibration of inelastic deformation capacity from the abovementioned database inherently accounts also for the deformability contributions due to shear and, above all, fixed-end-rotation, i.e. the rigid rotation at each element's end due to the slippage of longitudinal reinforcement from the adjacent element, which plays a very significant role, especially in elements with plain bars.

3.2.1 *Influence of the "fictitious compression" of beams related to the rigid floor diaphragms modelling*

Floor slabs in RC buildings are usually characterized by high stiffness in their own plane. This may be simulated by introducing rigid diaphragms that constraint mutual displacements between nodes of the same floor. On the other hand, beams are often modelled as one-dimensional elements connected at their ends to the floor nodes.

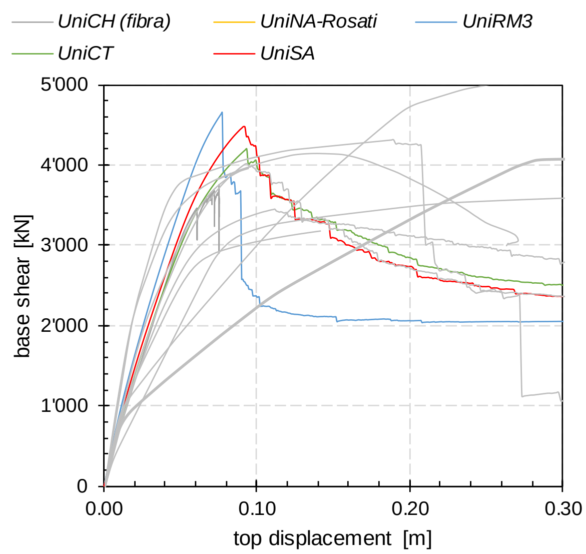

Figure 5. Capacity curve along Y-direction in models with rigid diaphragm / Curve di capacità in direzione Y in modelli con diaframma rigido.

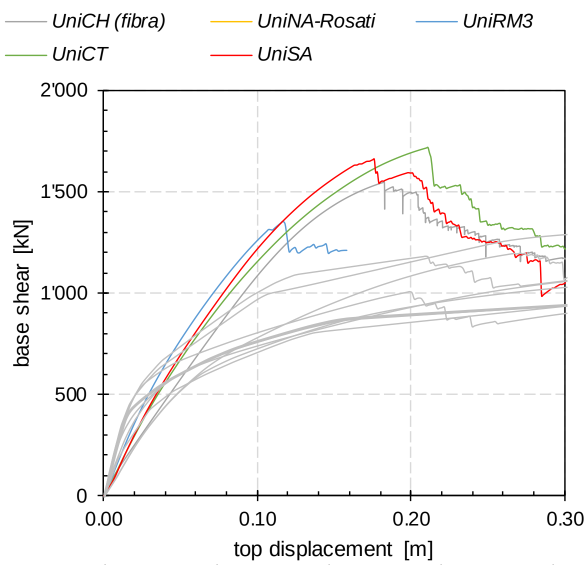

Figure 6. Capacity curve along X-direction in models with rigid diaphragm / Curve di capacità in direzione X in modelli con diaframma rigido.

Figure 5 and Figure 6 report the capacity curves along the Y- and X-direction, respectively, derived by the UR which simulate the RC floors by means of rigid diaphragms. The curves are compared with the ones already depicted in Figure 3 and Figure 4 referring to the basic models used by UR. Such curves are reported in grey in Figure 5 and Figure 6 with the aim of highlight the effects obtained as a result of the fictitious compression induced in beams by the rigid constraint. The interaction between beam elements and the rigid diaphragm may distort the response of the beams in which the elongation under flexural actions of the center fiber (as after cracking the neutral axis shifts from the center of the section and the longitudinal axis of the beams tends to elongate) is prevented by the rigid diaphragm which in turn transmits a fictitious (compression) axial force to the beam. This axial force leads to the overestimation of the bending moment resistance, which in turn determines an inaccurate prediction of the structural response and modify the collapse mechanism. Such an effect is more relevant along the X-direction where there are not seismic resistant frames.

The drawback described above may be overcome by introducing an additional element in the FE model, named "axial buffer element" (Barbagallo et al., 2018), which is a Zero-Length element that connects one end of each beam to the corresponding node in the rigid diaphragm. The axial stiffness of the buffer element is very low in order to allow the beams to deform axially freely and prevent the development of the fictitious axial force. Instead, the shear and flexural stiffnesses of this element are very high to restore the continuity of the structure and ensure the transmission of shear force and bending moment.

The nonlinear analysis executes by OpenSEES numerical models subjected to seismic excitation shows that the use of the buffer element leads to a more reliable estimation of the bending moment resistance of the beams and to a more accurate prediction of the seismic response of the structure.

A different approach is adopted by UniSA in order to reduce the development of the fictitious axial force. Specifically, floor slabs are modelled with elastic equivalent truss elements and, as it emerges from Figure 5 and Figure 6, such a simulation leads to accurate prediction of the seismic response of the structure which results to be close to the one obtained by using the more refined "axial buffer element".

4 CONCLUSIONS

The present work outlines the non-linear modeling techniques adopted by the WP2 group of the ReLUIS Research Project 2017 in order to perform pushover analysis on representative numerical models of an existing structure not designed according to the principles of Capacity Design. The collaboration between the research units (UR) about the definition of the relevant input data, made it possible to minimize the uncertainties (e.g. geometric dimensions, applied loads, mechanical properties of the materials) and obtain a realistic assessment of the numerical model associated with the use of different modeling techniques.

Therefore, the work is intended as a tutorial guide to technicians working on seismic assessment of existing RC buildings through non-linear analyses. The differences in the results obtained using different modeling techniques and different seismic analysis codes show a non-negligible sensitivity to the numerical prediction of the structural response both in terms of resistance and deformation capacity. Relevant is the study of the effect on the structural response of the floor stiffness simulation by diaphragm constraint, which can introduce a fictitious compression in beams altering their bending strength and therefore led to unrealistic estimations of both the structure response and the collapse mechanism. / Il presente lavoro illustra le tecniche di modellazione non-lineari adottate dal gruppo WP2 del Progetto di Ricerca ReLUIS 2017 al fine di eseguire analisi pushover su modelli numerici rappresentativi di una struttura esistente non progettata secondo le moderne regole di progetto ispirate al Capacity Design. La collaborazione fra le unità di ricerca (UR), nella fase di inserimento dei dati di input, ha permesso di ridurre al minimo le incertezze insite nelle scelte dell'analista (dimensioni geometriche, carichi applicati, proprietà meccaniche dei materiali) ottenendo una verifica realistica dei modelli associati all'uso di differenti software di analisi.

Il lavoro pertanto vuole essere un utile documento per i professionisti impegnati nelle procedure di verifica di edifici esistenti in CA a telaio tramite analisi non lineari agli elementi finiti. Le discrepanze fra i risultati ottenuti con differenti tecniche di modellazione e differenti software di calcolo hanno mostrato una non trascurabile sensitività sulla previsione numerica della risposta strutturale sia in termini di resistenza che in termini di capacità di spostamento. Di rilievo lo studio della modellazione della rigidezza dei solai con il vincolo di diaframma rigido che genera l'insorgere di azioni fittizie di compressione nelle travi comportando una sovrastima del loro momento resistente e, di conseguenza, stime non accurate della risposta strutturale e del meccanismo di collasso.

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