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Calibration of a Simplified Model for Dynamic Response Assessment of Infilled RC Buildings

Marco Gaetani d'Aragona[a], Maria Polese[a], Andrea Prota[b]
[a] *Dipartimento di Strutture per l'Ingegneria e l'Architettura, Via Claudio 21, 80125 Napoli, Italy*

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ABSTRACT
The prediction of the engineering demand parameters, such as interstorey drifts and peak floor accelerations due to a seismic event, represents a necessary stage for the assessment of direct economic losses in existing buildings. Generally, performing non-linear dynamic analyses on highly refined finite element models would be the most effective method for the estimation of such parameters. However, these procedures require a level of detail that is not employable on the large scale because of the required computational effort.

This work presents a simplified model, that can be adopted for the rapid evaluation of the engineering demand parameters in infilled frames subject to seismic actions. The proposed simplified model consists of a system of masses concentrated at each storey, connected by means of non-linear elements that properly describe the interstorey behavior. The procedure adopted to construct the simplified model of a multi-span multi-storey frame is described in detail. The interstorey non-linear envelope is defined by properly assembling the envelope of individual members under the simplifying hypothesis that rotation at the end of the columns are restrained. The hysteretic behavior of non-linear elements is calibrated for each storey based on the response of a 3D building modeled adopting a refined finite element model, and using the results of non-linear cyclic pushover analysis. The calibration is performed by adopting a multi-objective optimization procedure that involves the use of a Genetic Algorithm. The results of the proposed model are compared with those obtained by finite element analysis of a reference building for different intensities. The proposed model can be easily applied to carry out simplified numerical analyses useful for the assessment of direct economic losses at the large scale.

1 INTRODUCTION

The computation of seismic risk is one of the primary steps towards the assessment of earthquake consequences in a region of interest. In particular, the PEER performance-based earthquake engineering framework (Porter 2003) adopts economic losses as one of the primary metrics to quantify and communicate the risk to stakeholders. The general framework proposed in PEER relies on the structural analysis to calculate the seismic performances of a structure, expressed in terms of engineering demand parameters (EDPs) such as interstorey drift ratios (IDRs) and peak floor accelerations (PFAs). The distribution of EDPs throughout the structure is used to predict the damage in structural, non-structural components, and building contents.

Ideally, EDPs can be retrieved by performing time-consuming nonlinear time history analyses on very refined Finite Element Models and by performing a component-based damage assessment to compute expected losses (Aslani and Miranda 2009, Ramirez et al. 2012). With the aim to simplify the estimation for practitioners, a simplified approach for component-based damage assessment, adopting pushover analyses and capacity spectrum method, could be adopted (Gaetani d'Aragona et al. 2018a). However, with the purpose of estimating seismic risk at the large scale, there is the need to develop more simplified approaches for estimating IDRs and PFAs

This paper introduces a simplified model that can be employed for the rapid assessment of EDPs in existing infilled moment resisting frames subjected to seismic loadings. Starting from an idea already introduced in (Mollaioli et al. 2009, Xiong et al. 2016, Gaetani d'Aragona et al. 2019a), a nonlinear simplified dynamic MDOF model is proposed. The proposed model consists of a system of masses lumped in each storey, connected by means of nonlinear link elements. The proposed model has been demonstrated to predict with sufficient accuracy the behavior of reinforced buildings at both building scale and large scale (Xiong et al. 2017). However, the method requires an appropriate calibration depending on the building typologies to be assessed (Gaetani d'Aragona et al. 2019a). In particular, with the purpose of estimating seismic risk at the large scale, there is the need to develop simplified approaches for estimating IDRs and PFAs in existing RC buildings in European-Mediterranean countries, suitably accounting for the presence of infills (Gaetani d'Aragona et al. 2019b, Polese et al. 2018)

In the next section the case-study building for which the calibration procedure is performed is introduced. The geometrical and mechanical properties of all structural members is presented along with the definition of element envelope. In section 3, the refined finite element model that is adopted both to calibrate the hysteretic envelope of the nonlinear link elements and to compare the simplified model response is presented. Section 4 describes the construction of the interstorey backbone curve and the calibration of the hysteretic behavior via Genetic Algorithm and section 5 presents the results of comparison between the simplified and the refined building model for a ground motion bin.

2 CASE-STUDY BUILDING

The case-study building consists of a 6-storey gravity load designed (GLD) building constructed in Italy before '70s. The structural model consists of an infilled moment-resisting frame.

The structural and geometrical model for the frame is obtained by means of a simulated design procedure described in (Verderame et al. 2010). Complying with the building codes (e.g., RDL. n.2229/1939, CMLLPP n.1472 23/5/1957) and the design practice in force at the time of construction (e.g., Polese et al. 2011), the structural elements are firstly dimensioned. The longitudinal reinforcement in columns is designed with reference to minimum longitudinal reinforcement geometric ratio prescribed by code for gravity load designed buildings, while the longitudinal reinforcement for beams is designed considering envelope moments deriving by limit load combination schemes according to construction practice. According to the Italian construction practice in force before '70s, the moment resisting frames are plane and formed by columns and deep beams only in one direction, since deep beams are mainly deputed to absorb gravity loads and are placed only in the direction perpendicular to the one-way slabs, while in the other direction the structure is formed by columns and beams that are embedded in the thickness of the horizontal slab; thus, only the perimeter frames are characterized by the presence of deep beams (Polese et al. 2011). Finally, the perimeter frames are infilled, and it is supposed that the area of openings for the infills correspond to 20% of the infill area for each panel.

2.1 *Geometrical and mechanical properties*

For the considered building plan rectangular shape, regular both in plan and in elevation is assumed. The building is assumed having 3 bays in the longitudinal and 2 bays in the transversal direction. The bay span in both the longitudinal and transversal direction are a_x=a_y=4.0m and an interstorey height of a_z=3.0m is assumed.

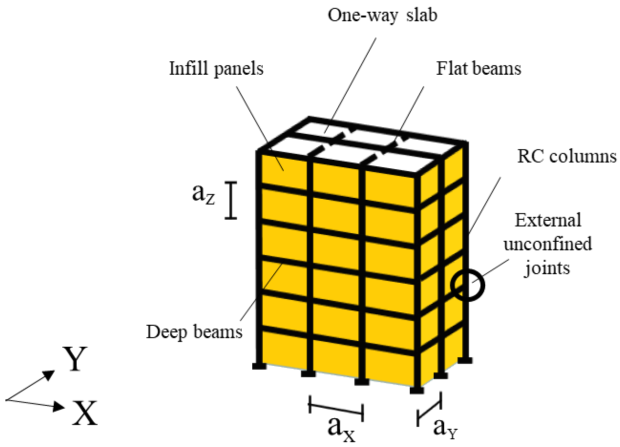

Figure 1. Sketch of the case-study building

Adopting the simulated design procedure described above, the structural model for the archetype building is obtained. Longitudinal frame columns dimensions are 30×30cm at each storey, except that for interior columns of the intermediate frame that are 40x30cm at first storey and 35x30cm at the second storey. Deep beams are 40x30cm at each storey for perimeter longitudinal and transversal frames and 45x30cm for the intermediate longitudinal frame, while flat beams (only in the transversal direction) are 20x35cm for every intermediate transversal frame.Column reinforcement varies between 6ϕ16 for first storey interior columns and 4ϕ12 for first storey corner columns. For every column, shear reinforcement consisting of ϕ6 stirrups with 90-degree hooks and 30cm spacing is assumed. In every structural element, according to construction practice in force before '70s, plain rebars are adopted. Referring to the concrete and steel properties, a compressive concrete stress fc = 25 MPa and a steel tensile yielding stress f_y = 399MPa are chosen as representative values for GLD buildings constructed in the decade '62-'71. Infill panels are realized with a double layer of hollow clay bricks having (80+120) mm thickness. The elastic shear modulus is assumed equal to G_w = 1350 MPa. The E_w is assumed equal to $0.4G_w$, while cracking strength of the masonry is considered linearly dependent on G_w according to boundary values indicated in (CMLLPP n.617 2/2/2009) .

2.2 *Definition of element envelope*

For RC members, a multi-linear moment-rotation envelope is built with cracking and yielding as initial characteristic points. Moment at yielding (M_y) is calculated according to the simplified formulation proposed by Biskinis and Fardis 2010, while the rotation at yielding (θ_y) is identified by M_y and the secant stiffness (EI_y) provided by Haselton et al. 2008. The brittle failure of non-conforming elements may significantly impact the global behaviour of existing structures (Gaetani d'Aragona et al., 2017). Thus, for each column, the expected failure mode is determined by comparing the yielding shear (V_y), calculated as the ratio between M_y and the shear span of the column (L_v), and the shear strength (V_n) calculated according to EC8 (2005). The L_v is assumed equal to one half of the column height. Despite V_n may be significantly influenced by the column axial load variation due to horizontal loads, here only initial gravity loads are considered (Miano et al.2017). Depending on the ratio V_y/V_n, three possible failure modes are expected: flexure, flexure-shear or pure shear failure. If $V_y/V_n < 1$ for any value of ductility demand, RC column is expected to fail in flexure and a three-branch back-bone is built, in which the ultimate rotation capacity corresponds to the ultimate chord rotation for ductile members (Panagiotakos and Fardis 1998). If $V_y/V_n \geq 1$, the column is expected to fail in shear or flexure-shear. In the latter case a four-branch backbone is built that includes a degrading slope identified by the ultimate shear and the ultimate axial rotation capacities evaluated according to the simplified relationship proposed in Aslani and Miranda (2009). Due to the presence of plain reinforcement bars, longitudinal bar-slip effect in columns and beam-column joints cannot be neglected. Thus, the bar-slip effect is accounted adopting the model proposed in (Sezen and Mohele 2003, Sezen and Setzler 2008)

Another characteristic of existing GLD RC frames is to the total lack of transverse reinforcement in the joint region, thus the possible joint failure should be considered. According to Pampanin et al. 2003, the joint shear capacity in beam-column joints with inadequate structural detailing can be directly related to principal tensile/compressive stresses. The beam-column joint backbone is thus constructed adopting the principal stresses and the panel rotation limits proposed in (Pampanin et al. 2003, NZSEE 2017) transformed into the corresponding moment-rotation relationship directly derived from equilibrium considerations.

The behaviour of infill panels adopts the force-displacement envelope evaluated according to the model proposed by De Risi et al. 2018 in which the lateral force-displacement relationship is constructed depending on the geometry of the surrounding frame, and on both mechanical and geometrical characteristics of the infill masonry.

The mechanical characteristics of the masonry are expressed in terms of elastic shear modulus, G_w, Young's modulus, E_w, and shear cracking strength, $\tau_{cr.}$.Finally, the equivalent strut width, b_w, is determined according to Mainstone's formula (1971) depending on quantities reported above and on the moment of inertia, I_c, and Young's modulus, E_c, of columns. When openings are present in the infill panel, e.g., to accommodate windows or balconies, both the stiffness and the strength of the infill panel are reduced. The presence of the opening is considered introducing a reduction factor (Al-Chaar 2002) that modifies both the stiffness and the strength of the infill panel, where A_o is the area of openings and A_p the area of infill panels.

3 REFINED MODEL

A fixed-base three-dimensional finite element MDOF model developed using Opensees (2019) is used to simulate the seismic response of the building. Figure 2 shows a schematic representation of the generic frame of the building. The frame elements are modeled using lumped plasticity elements consisting of two inelastic rotational hinges connected in series by an elastic beam-column element. The inelastic behavior of beams and columns is conveniently characterized by a multilinear moment–rotation relationship in the plastic hinges, described by means of a set of characteristic points as indicated in §2. As noted in (Polese et al. 2013, Ibarra and Krawinkler, 2005), since the frame members are modeled as an elastic element connected in series with rotational springs at either end, the stiffness of these components must be modified so that the equivalent stiffness of this assembly is equivalent to the stiffness of the actual frame member. Following the approach proposed in (Ibarra and Krawinkler, 2005), the rotational springs are made "n" times stiffer than the rotational stiffness of the elastic element in order to avoid numerical problems. To ensure the equivalent stiffness of the assembly is equal to the stiffness of the actual frame member, the stiffness of the elastic element must be "(n+1)/n" times greater than the stiffness of the actual frame member. Similar considerations must be accounted for in the definition of the degrading branch slope of flexure-shear critical members. The hysteretic behavior of column/beam elements is modeled adopting the Pinching4 material (Lowes et al, 2004) with hysteretic rules proposed by (Lee and Han 2018) for old reinforced concrete columns.

To account for bar-slip effect, a zero-length bar-slip plastic hinge is added at the extremities of either ends, no hysteretic degradation is assumed for these elements.

The joint behavior is modeled using the "scissor" model (Alath and Kunnath 1995), which includes the pinching hysteretic behavior to account for the non-linear shear deformation of the joint. The joint constitutive model adopts the backbone curve proposed in NZSEE (2017) with equivalent moment-rotation relationship obtained using the formulation proposed in Celik and Ellingwood (2008). The hysteretic behavior for beam-column joints is simulated adopting the Pinching4 material with hysteretic rules proposed in Hassan (2011). The masonry infills are modelled by means in-plane equivalent diagonal struts carrying loads only in compression. The hysteretic behavior for these elements is simulated adopting a Pinching4 material with parameters defining the cycling degradation proposed in Lima et al. (2014). Note that such a simplified model is unable to simulate the local effects on the columns due to the presence of the masonry infills since it does not properly reproduce realistic moments and shear forces in the columns. For this reason, this model may lead to overestimate the strength and ductility capacity of the building (Gaetani d'Aragona et al. 2019c)

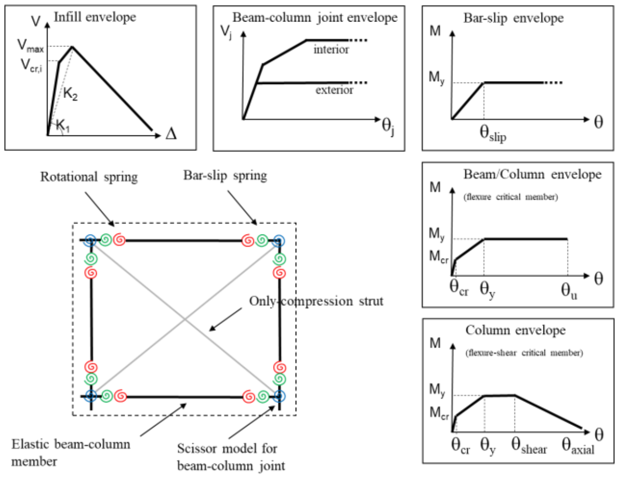

Figure 2. Model representation of the generic frame for the refined model

Finally, the presence of RC one-way slabs is simulated by means of an elastic shell element. Classical Rayleigh damping is adopted with a value of 5 %.

4 SIMPLIFIED MODEL

To allow the prediction of the building response in large-scale seismic simulations, a multiple degree-of-freedom (MDOF) non-linear shear model (NLSM) is adopted (see Figure 3). In particular, each storey is discretized into a nonlinear shear spring with the following assumptions: 1) The model assumes that the mass of each storey is concentrated on its elevation and represented by lumped masses; 2) The seismic response of the structure is dominated by interstorey shear deformations; 3) The building has regular planar layout.

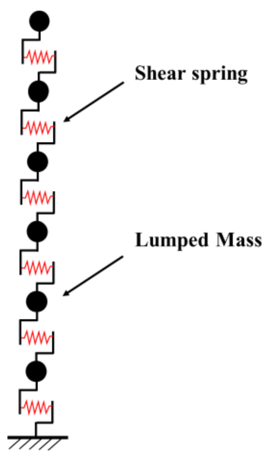

Figure 3. Non-linear Shear Model (NLSM)

Some authors (e.g. Xiong et al. 2017) have shown that NLSM are suitable to describe the nonlinear characteristics and failure modes of multistorey buildings properly capturing the damage concentration in each storey.

The elastic and inelastic properties of the shear springs are regulated by the properties of the infill wall and the frame. To determine the parameters in the interstorey hysteretic model, a method based on a simplified procedure to combine the behavior of multiple components is adopted (Gaetani d'Aragona et al. 2018b).

4.1 Construction of backbone curve

The interstorey backbone for the simplified model is defined in terms of shear-displacement response and is assembled considering that ends of the columns are restrained against rotation (Shear Type model), as already proposed in (Gaetani d'Aragona et al. 2018b, 2019a). This simplifying hypothesis allows to reproduce the seismic response of existing buildings with a reasonable degree of approximation.

Once that force-displacement/moment-rotation relationships is defined for each relevant member, these are transformed in the corresponding shear-displacement curves and then, considering the RC frames and infill elements at the same storey as acting in parallel, a multi-linear interstorey shear-displacement relationship is constructed for each storey. The comparison in terms of Pushover curve (1st mode proportional load pattern) between the refined finite element model (FEM) presented in §3 and the NLSM, in which a multi-linear back-bone for the interstorey shear-displacement envelope is adopted, is depicted in Figure 4.

As can be noted from Figure 5, the NLSM can accurately describe the global behavior of the FEM. The very low scatter between FEM and NLSM is related to assumption of shear-type model. However, this assumption to calibrate the interstorey shear-displacement relationship does not introduce significative bias. In fact, the presence of slab, that is explicitly simulated in the FEM, introduces a partial restrain against beam rotations limiting out-of-plane the inflection of slab making the shear-type assumption partially valid.

A four-linear backbone curve is adopted to represent the nonlinear behavior each storey. The adoption of a four-linear backbone curve model can accurately represent the interstorey behavior of a structure with acceptable modeling complexity and computational accuracy.

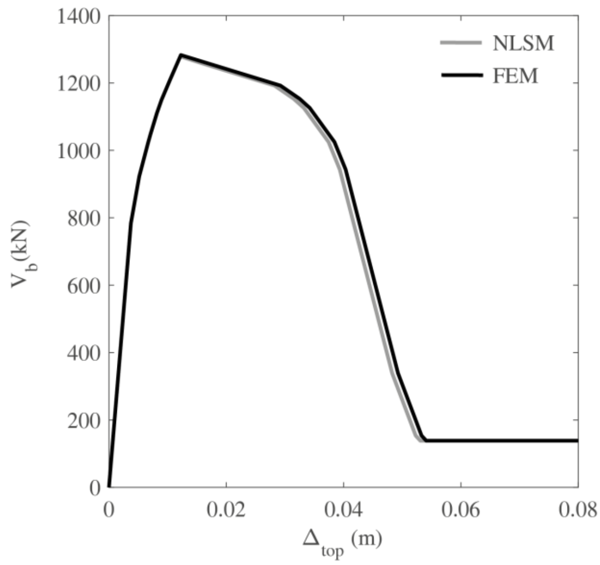

Figure 4. Pushover curve performed on the Refined FEM (black) and on the NLSM (gray) adopting multi-linear back-bone for interstorey backbone.

Figure 5 shows the backbone curve obtained with the proposed procedure and the corresponding four-linear backbone adopted to approximate the interstorey shear-displacement (V-Δ) curve in the NLSM.

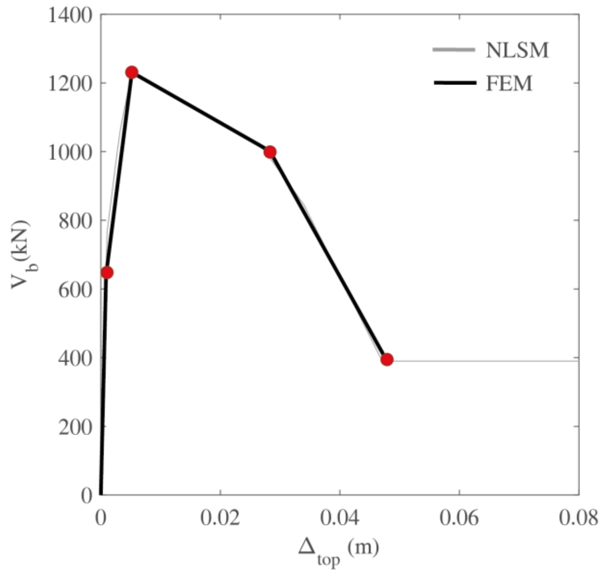

Figure 5. Interstorey shear-displacement curve for the second storey obtained with the proposed simplified procedure (gray) and four-linear backbone approximation for NLSM (black).

4.2 Hysteresis loop calibration

The NLSM requires the calibration of a nonlinear interstorey hysteretic model for each storey of the building. While the backbone curve can be easily calibrated according to the simplified procedure indicated in §4.1, the calibration of hysteretic rules represents a more complex issue having to account for different degradation modes of multiple components in the same storey. The Pinching4 model [30] is here employed to simulate the pinched response and cyclic degradation of strength and stiffness in each storey. The Pinching4 material requires the definition of 21 parameters that govern stiffness, and strength deterioration. However, in order to simplify the calibration procedure, it is assumed that the pinched behavior is symmetric in both the positive and the negative direction, and that the deterioration only depend on the energy-dependent terms. This way, the number of parameters to be calibrated is equal to 9.

For each storey, the calibration is performed adopting a Finite Element model composed of a single link element, with one end point fixed and a displacement loading history applied to the other end. For each storey the backbone defined in §4.1 is adopted as monotonic envelope, and the interstorey hysteretic response is calibrated in order to match the reverse cyclic pushover displacement history performed for the generic storey of the refined FEM model described in §3.

The calibration of Pinching4 parameters has been carried out adopting a second generation Genetic Algorithm (GA) optimization procedure (Deb et al. 2002) that has been implemented in Matlab©. The GA allows the multi-objective optimization based on a selection process that mimics biological evolution. Starting from an initial population of individual solution, the GA repeatedly modifies the population randomly selecting individuals from the current population and using them as "parents" to produce "children" for the next generation. By simulating also "crossover" and "mutations", the GA allows through successive generations to evolve toward an optimal solution, that for the multi-objective optimization is represented by a Pareto optimal solution. A solution is referred to as Pareto optimal if it is not dominated by any other solution. The scope of the GA is the minimization of the fitness function. In this study a multi-objective calibration problem is set considering three objective function:1) Minimize the scatter between the cumulated energy of the FEM model and the single link element for each storey; 2) Minimize the scatter between the Force history of the FEM model and the link element for each storey; 3) Minimize the scatter between the hysteretic damping of the FEM model and the single link element for each storey.

5 COMPARISON OF THE NLSM WITH FEM

The main goal of this study is to establish a general framework for the calibration of the hysteretic behavior of a simplified model that is able to accurately represent the response of a structure with a low computational effort and sufficient accuracy in prediction of EDPs. To validate the reliability and accuracy of the proposed model and the calibration method adopted, the response of the calibrated NLSM is compared with that of the refined FEM the far field record set included in FEMA-P695 (ATC 63, 2008). In particular, the results of the two models are compared for a single intensity equal to 0.2g.

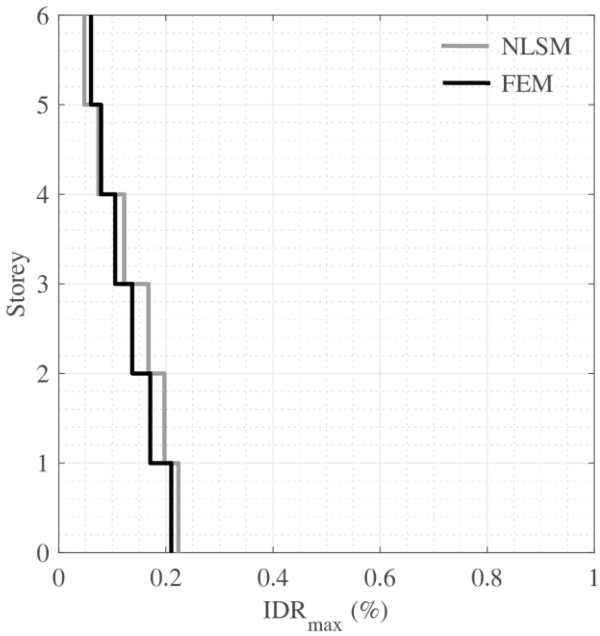

Figure 6. Median Interstorey Drift Ratio profile for the FEMA-P695 ground motion bin obtained with the Non-linear Shear Model (gray) and Finite Element Model (black).

Figure 6 shows the comparison between the NLSM and the FEM in terms of median Interstorey Drift Ratio for the FEMA-P695 ground motion bin for 0.2g. The comparison evidences a very good agreement between the two models.

In particular, the story at which the maximum IDR occurs and the distribution of deformation along the height is well represented by the simplified model. Note also that, due to the brittle behavior of infill panels, the model also experienced plastic deformations despite the low level of intensity. In terms of IDR, the maximum value is attained at the first storey, and the scatter between the NLSM and the FEM is about 5%, the maximum discrepancy between the two models occurs at the third storey, in which the difference in prediction is of about 20%.

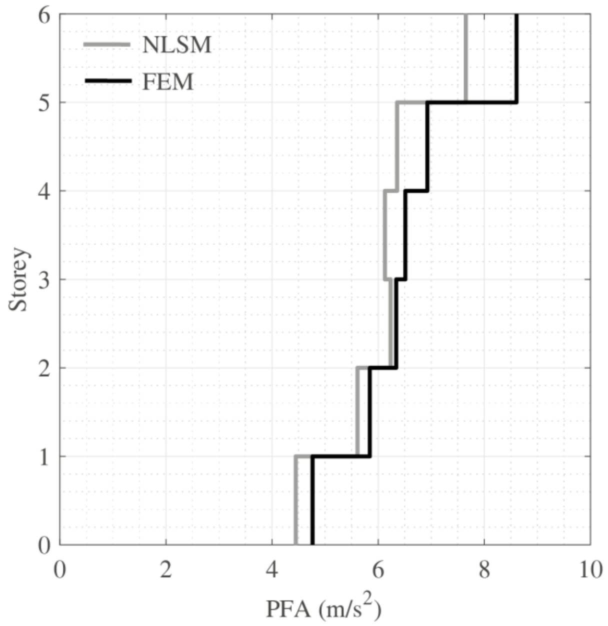

Figure 7. Median Peak Floor Acceleration profile for the FEMA-P695 ground motion bin obtained with the Non-linear Shear Model (gray) and Finite Element Model (black).

The results in terms of median Peak Floor Acceleration are shown in Figure 7 for the NLSM and the FEM. In this case, the distribution of PFAs is still well represented. In particular, the maximum PFA occurs at the higher storey, and the maximum scatter between two models is of 21%, confirming the same level of approximation noted for the IDRs. However, in this case the NLSM underpredicts the maximum PFAs.

6 CONCLUSIONS

A simplified model and the associated parameter calibration procedure are introduced for the rapid assessment of engineering demand parameters in existing infilled reinforced concrete buildings. The proposed simplified model consists of a multiple degree-of-freedom nonlinear shear model.

The main scope of this work is to set a framework for the calibration of the hysteretic behavior of a simplified model to accurately represent the response of a structure in prediction of engineering demand parameters with a low computational effort and sufficient accuracy at large scale. The procedure to construct the simplified model for a multi-span, multi-storey reinforced concrete frame is described. In particular, the interstorey nonlinear envelope is assembled starting from the single component behavior under the simplified hypothesis that the ends of the columns are restrained against rotation. The hysteretic behavior of nonlinear links is performed adopting a multi-objective Genetic Algorithm procedure that employs the results of reverse static pushover performed on a more refined finite element model.

The calibration procedure is demonstrated for an existing six-story building obtained via simulated design procedure. The results of the proposed method are compared with those of a more refined finite element model that properly accounts for the typical features of existing reinforced concrete buildings. The simplified model showed a good agreement in prediction of engineering demand parameters when compared to the more refined model.

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