

# Probabilistic seismic performance assessment of infilled RC frame buildings



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## SUMMARY:

A methodology for the probabilistic seismic performance assessment of infilled reinforced concrete frames with consideration of shear failure of columns is presented. It is based on an iterative pushover procedure, which involves pushover analysis, post-processing of the analysis results using limit-state checks of the components, and model adaptation if shear failure of columns is detected. By means of an example of a four storey building it is shown that the shear failure of columns due to the effects of masonry infill may have a significant impact on the seismic response of reinforced concrete frames with masonry infills. The damage in the structural elements and the collapse mechanism differ significantly to those where the shear failure of columns is neglected in the analysis. Consequently, the mean annual frequency of limit-state exceedance was significantly larger than that estimated without consideration of the shear failure of the columns.

*Keywords: performance-based earthquake engineering, seismic risk, iterative pushover procedure, shear failure, infilled RC frame*

## 1. INTRODUCTION

Reinforced concrete frames with masonry infills are commonly found in most parts of Europe and other places around the world. Last event, which triggered new research associated with seismic behaviour of infilled RC frames, was the L'Aquila earthquake, which once again highlighted the vulnerability of such buildings (Braga et al., 2010; Verderame et al., 2010).

Simulation of seismic response of infilled RC frames is difficult not only because it requires a model of an entire structure but also due to some specific features of this structural system. Thus, infilled RC frames are usually modelled by means of so-called macro-models, which consist of beam/column elements and diagonal struts, representing the masonry infills. A comprehensive overview of variants of diagonal-strut models for infilled frames was prepared by Crisafulli et al. (2000). The simplest model of an infilled frame consists of a frame model and single compressive diagonal struts, which are connected to the frame through the beam-column joints. The advantage of this model in comparison to other multi-strut models is in its simplicity, since only compressive diagonal struts are added to the frame model, without increasing the number of joints or beam/column elements. Alternatively, infilled RC frames can be modelled by means of four-node element (Crisafulli and Carr, 2007) or by even more demanding FEM models (e.g. Alam et al., 2009; Combescure and Pegon, 2000). All these different types of structural models have their pros and cons, but, in general, it is difficult to predict the response of infilled RC frames since their response is extremely nonlinear.

The main disadvantage of the simplest single-strut model of the infill is in its inability to simulate directly the internal forces acting on the columns, since the contact between the column and the frame is neglected. Therefore, an iterative pushover-based procedure involving model adaptation is introduced in order to improve the capability of simulations based on simplified nonlinear models of the infilled RC frames, which involve single-strut model of the infill. The objective of the proposed

procedure is to approximately incorporate the effects of masonry infills on the shear demand of the columns, to simulate the potential shear failure of columns, and to quantify these effects through a probabilistic seismic performance assessment procedure based on fragility analysis and estimation of the mean annual frequency of limit-state exceedance.

In the paper, the simplified nonlinear models of the infilled RC frames, the iterative pushover procedure and the risk assessment method are briefly summarized. In the second part of the paper the proposed probabilistic seismic performance assessment of infilled frames is demonstrated by means of the example of a four storey building.

## 2. DESCRIPTION OF SIMPLIFIED NONLINEAR MODELS OF INFILLED FRAMES

The columns and beams of the simplified nonlinear model were modelled by one-component lumped plasticity elements, which consisted of an elastic column/beam elements, each with two inelastic rotational hinges at its ends, whereas the infills were modelled by in-plane equivalent diagonal struts, carrying loads only in compression. The moment-rotation relationship in the plastic hinges of the beams and columns was determined by means of three characteristic points, i.e. the yield point, the maximum strength and the near-collapse point (NC). This model is supported in the PBEE toolbox (Dolsek, 2010), which allows rapid generation of simple nonlinear models and the analysis of infilled RC frames in conjunction with OpenSees (2009). In the case of the beams, the plastic hinge was used for the major axis of bending only, whereas in the case of the columns, two independent rotational hinges for bending about the two principal axes were defined. The yield and the maximum moments were calculated by moment-curvature analysis, taking into account the axial forces in the columns due to gravity loads, whereas the axial forces in the beams were considered to be equal to zero. The stiffness and strength of the beams were determined taking into account effective slab widths according to Eurocode 2 (CEN, 2004a), where the zero moment point is assumed to occur at the mid-span of the beams. The NC point represented a local near-collapse limit state of the columns or beams was defined by the ultimate rotation  $\theta_{u,f}$ , which corresponded to a 20 % reduction in the maximum moment. In the case of the columns,  $\theta_{u,f}$  was estimated by means of the Conditional Average Estimator (CAE) method (Peruš et al., 2006), which involves a regression analysis that is applied to the two experimental databases, whereas in the case of the beams  $\theta_{u,f}$  was determined by using the formula defined in EC8-3 (CEN, 2005).

The masonry infills were modelled by means of diagonal struts. This model, although relatively simple, is widely used since it is numerically stable and computationally efficient. Its main uncertainty is related to the definition of the force-displacement relationship of the diagonal strut. In the literature, many different proposals for the determination of the stiffness and strength of masonry infills have been made. Some of them were summarized and reviewed in a previous paper by Dolšek and Fajfar (2008). In this study the model of diagonal is the same as that used in previous study (Celarec et al., 2011). The force-displacement relationship of the diagonal strut consisted of four branches and is based on the approach defined by Panagiotakos and Fardis (1996), and Fardis (1996). The parameters of the force-displacement relationship of the diagonal struts, if measured in the horizontal direction, are:

- the shear strength at cracking, calculated as  $F_{cr} = \tau_{cr} \cdot A_w$ , where  $A_w$  is the horizontal cross-sectional area of the infill, and  $\tau_{cr}$  is its diagonal cracking strength,
- the maximum shear strength, estimated as  $F_{max} = 1.3 \cdot F_{cr}$ ,
- the initial stiffness of the infill, determined as  $K_{el} = G_w \cdot A_w / h_w$ , where  $G_w$  is the shear modulus of the masonry infill,
- the secant stiffness to the maximum strength  $K_{sec}$  estimated according to Mainstone's formula (1971),
- the negative post-capping stiffness of the infill  $K_{deg} = -\alpha \cdot K_{el}$ , with parameter  $\alpha$ , which has to be assumed in the analysis and is based, for example, on experimental results (Fardis, 1996). The range of the values of  $\alpha$  should be between 0.01 and 0.1, although the latter is an unrealistically high value signifying very brittle collapse of the infill panel. In the study,  $\alpha$  was arbitrarily assumed to have a value of 0.05.

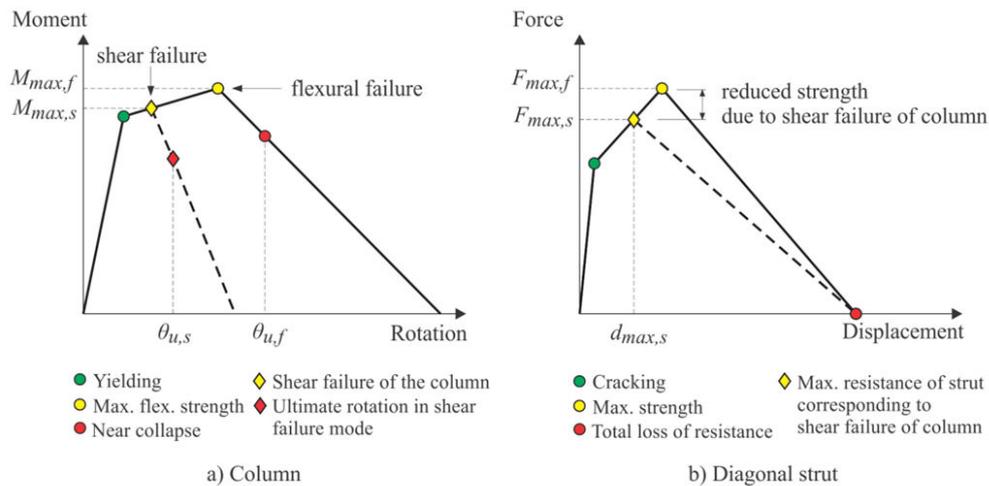
The total shear demand in a column ( $V_{D,t}$ ), which cannot be directly simulated by the single-strut model, is calculated as the sum of the shear force in the column resulting from the analysis ( $V_{D,a}$ ) and the additional shear force, induced by the masonry infill ( $V_{D,i}$ ):

$$V_{D,t} = V_{D,a} + V_{D,i}, \quad (2.1)$$

$$V_{D,i} = \gamma_{ci} \cdot N_s \cdot \cos \theta, \quad (2.2)$$

where  $N_s$  is the axial force in the equivalent diagonal strut and  $\theta$  is inclination of the diagonal strut with respect to the horizontal axis. The model parameter  $\gamma_{ci}$  defines the proportion of the force, which is transferred from the infill to the column. This parameter depends on the geometry, elastic and shear modulus of the frame, and on those of the masonry infill, as well as on the level of deformations induced in the structure. A value of  $\gamma_{ci}$  can be approximately assumed according to the definition of different multi-strut approaches, which allow direct simulation of the internal forces in columns (Crisafulli, 1997; Verderame et al., 2010). For example, Verderame et al. (2010) used a three-strut model of the infill, suggesting that 50 % of the total stiffness and strength of the infill are supplied by the central diagonal in compression, whereas the remaining stiffness and strength of the infill should be distributed along the other two (offspring) diagonals in compression. This means that, in the case of three-strut model, the additional shear force induced in the columns is equal to approximately 25 % of the horizontal projection of the sum of the axial forces in all three diagonals in compression. Furthermore, based on the results of an analysis, performed with a more refined FEM model, Combescure (2006) found that the additional maximum shear force in the column is equal to about 64 % of the shear force in the masonry infill. By means of numerical simulation he showed that this value is almost independent of the characteristics of the masonry infill and the type of the frame model (linear or non-linear). Base on a literature review it can be concluded that determination of parameter  $\gamma_{ci}$  is quite uncertain, since different authors have suggested significantly different values, starting from approximately 0.25 up to 0.64. In the example case study,  $\gamma_{ci}$  was assumed 0.5.

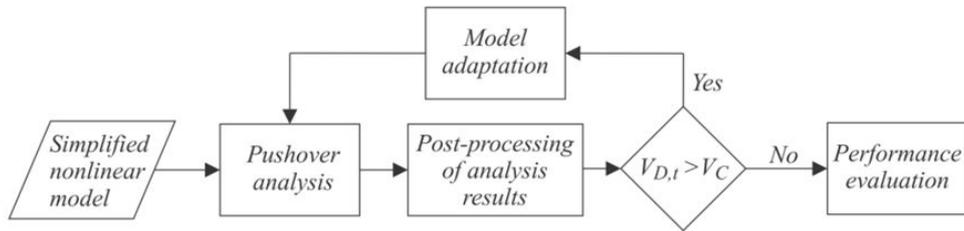
Whenever the shear demand in one of the columns exceeds the corresponding capacity, which was estimated according to the procedure proposed in Eurocode 8-3, it was assumed that the maximum moment has been attained in the column. If, therefore, the deformations are further increased, negative post-capping stiffness is adopted in both plastic hinges of the columns (Fig. 2a). This post-capping stiffness corresponds, in the case of shear failure of the column, to 80 % of the maximum moment in the column and the rotation capacity  $\theta_{u,s}$ , which is typical for the case if a shear-failure is observed in a column, and is assumed to be equal to the value suggested by Zhu et al. (2007). It was additionally assumed that the infill is unable to resist larger forces than those attained at the moment when shear failure occurs in the adjacent column (see Fig. 2b).



**Figure 2.1.** The moment-rotation relationship of a) the columns and b) the equivalent diagonal strut.

### 3. ITERATIVE PUSHOVER PROCEDURE AND RISK ASSESSMENT

The proposed pushover-based procedure involves post-processing of the analysis results and model adaptation, as is schematically presented in Fig. 3.1. Simulated and non-simulated failure modes are checked by post-processing of the analysis results in order to link the damage in the columns, beams and masonry infill at the element level to the top displacement and base shear, which define the pushover curve. Column shear demand ( $V_{D,i}$ ) and capacity ( $V_C$ ) is estimated based on the procedure described in Section 2. In the case when the column shear demand does not exceed the corresponding shear capacity, the results of the pushover analysis can be further used for performance evaluation. On the other hand, if it is indicated that  $V_{D,i} > V_C$  in at least one column, then the model is adapted as described in Sections 2, and the pushover analysis is performed. These steps are repeated until all failure modes are properly simulated by it. Note that each model is adapted so as to be capable of approximately simulating only that shear failure of the column which is detected at the lowest top displacement. The described iterative procedure was implemented in the PBEE toolbox (Dolsek, 2010), which was previously extended by in order to be applicable to the seismic performance assessment of infilled frames, modelled by a single-strut model (Ricci, 2011; Celarec et al., 2011).



**Figure 3.1.** Flowchart of the iterative pushover analysis procedure for the seismic performance assessment of infilled frames.

Infilled frames are good examples of structures which vibrate primarily in the fundamental mode, and they often collapse due to the concentration of damage in one storey in the lower part of a building. It is therefore very likely that simplified nonlinear methods, such as the N2 method (Fajfar, 2000; Dolšek and Fajfar, 2004), may give good estimate for the seismic performance of such structures. Since the basic N2 method does not allow to incorporate the effects of aleatory uncertainties, it was decided to use the nonlinear dynamic analysis of the equivalent SDOF model, which provides more accurate estimates of buildings response parameters, such as the median value and the logarithmic standard deviation of the intensity measure causing the violation of LS ( $i_{m50,LS}$  and  $\beta_{LS}$ ). The SDOF model, which was used to assess the fragility parameters ( $i_{m50,LS}$  and  $\beta_{LS}$ ) consisted of two parallel springs representing the frame and the infill. In addition to the SDOF model used in the previous study (Dolšek in Fajfar, 2004) for the determination of the inelastic spectra of infilled frames, the model adopted in this work made it possible to model negative stiffness after frame begins to degrade.

For reasons of simplicity, the fragility parameters were estimated only with consideration of aleatory uncertainty (record-to-record variability), since the objective of the paper was to demonstrate the potential error if shear failure of columns due to the effect of masonry infill is neglected in the probabilistic seismic performance assessment procedure. The limit-state ground motion intensity  $i_{m,LS}$  was thus calculated by performing incremental dynamic analysis (IDA) (Vamvatsikos and Cornell, 2002) for a set of ground motions. The sample of limit-state intensities was then used for estimation of the fragility parameters, for example, by using the maximum likelihood method. Finally, the mean annual frequency (MAF) of exceedance of a given limit was assessed according to simple closed-form solution (Cornell, 1996):

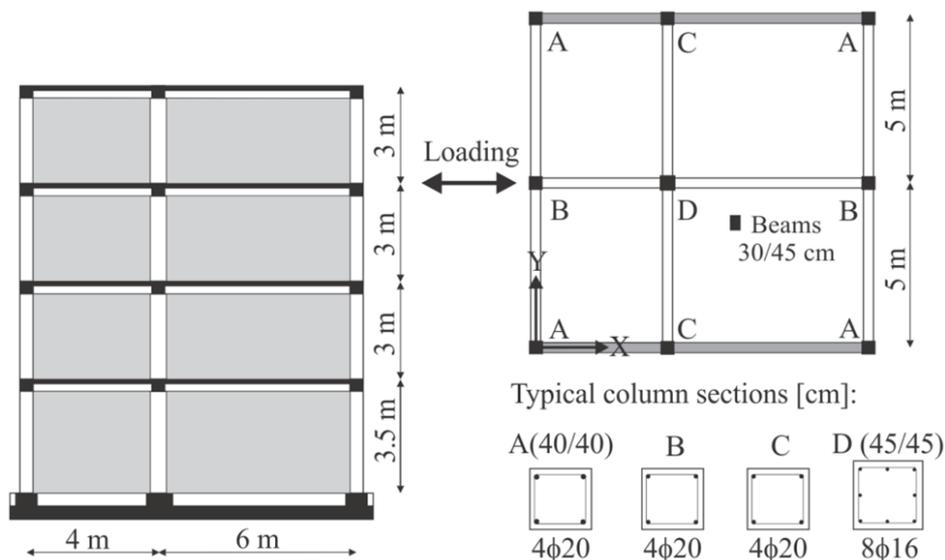
$$\lambda_{LS} = H(i_{m50,LS}) \cdot e^{\frac{1}{2} \cdot k_{LS}^2 \cdot \beta_{LS}^2}, \quad (3.1)$$

where  $k_{LS}$  is the slope of the hazard function  $H$ , which was simply estimated from the hazard maps for the two different return periods.

#### 4. FRAGILITY ANALYSIS AND PREDICTION OF MEAN ANNUAL FREQUENCY OF LIMIT-STATE EXCEEDANCE OF A RC FRAME WITH MASONRY INFFILLS

##### 4.1 Description of the selected infilled RC frame

A four-storey reinforced concrete frame with masonry infill was selected in order to demonstrate the proposed procedure for the seismic risk assessment of infilled frames. The elevation, plan view, and details of the cross-sections, showing the reinforcement in the columns, are presented in Fig. 4.1. The structure is a variant of the building which was originally designed according to the previous version of Eurocode 8-1 (CEN, 2004b), as a high ductility class structure (Fardis, 1996). The geometry of the building analysed here is the same as that of the original building, but the material characteristics of the concrete and that of the steel, as well as the amount of reinforcement in the columns and beams, were arbitrarily modified so that the columns became sensitive to shear failure, which was avoided in the original design of the building (Fardis, 1996).



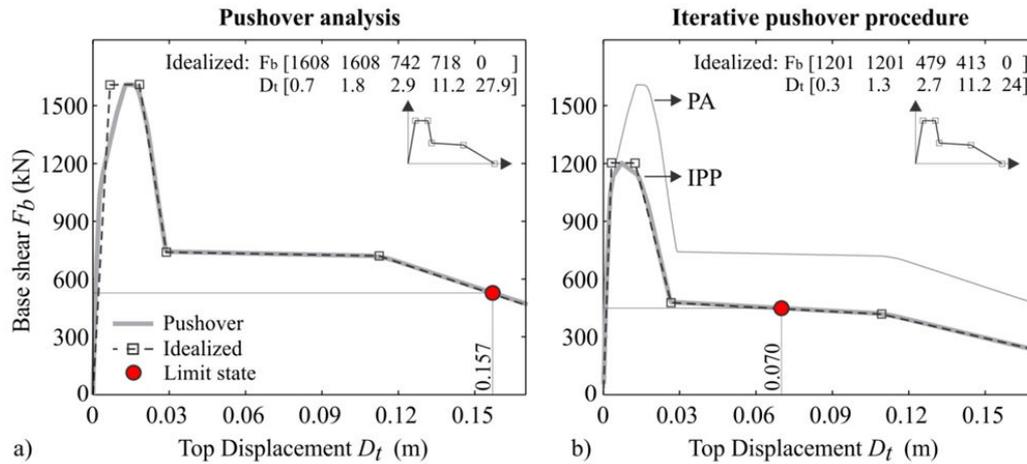
**Figure 4.1.** The elevation, plan view, and reinforcement in typical cross-sections of the columns.

##### 4.2 Pushover analysis and definition of near-collapse limit state

The pushover analysis of the structure was performed for the positive X direction of loading. The iterative pushover procedure (IPP) proposed in Section 3 was performed in addition to traditional pushover analysis (PA), which in this case neglects the shear failure of columns due to the limitations of the simplified nonlinear model. Four iterations were needed in the case of the IPP in order to adapt the simplified nonlinear model so that it is able to approximately take into account shear failure of the columns. Shear failure was first detected in the middle two columns in the second storey of the two external frames, followed by the shear failures of the middle two columns in the first and the third storey. At this stage of the analysis, the maximum strength was reached (Fig. 4.2). Beyond the corresponding displacement, the infills in the first storey started to degrade until the last infill in the thus-formed plastic mechanism totally collapsed. From here onwards, only the reinforced concrete frame resisted the lateral loads.

The pushover curves for the cases of the PA and of the IPP are presented in Fig. 4.2. As can be seen from the figure, the shear failures of the columns predominantly affected the strength of the structure.

For example, the maximum base shear observed in the case of PA ( $F_{max}=1600$  kN) was about 33% greater than that in the case of the IPP ( $F_{max}=1200$  kN). The deformation capacity did not decrease as drastically as had been expected since the columns in the middle frame (Fig. 4.1) did not fail in shear.



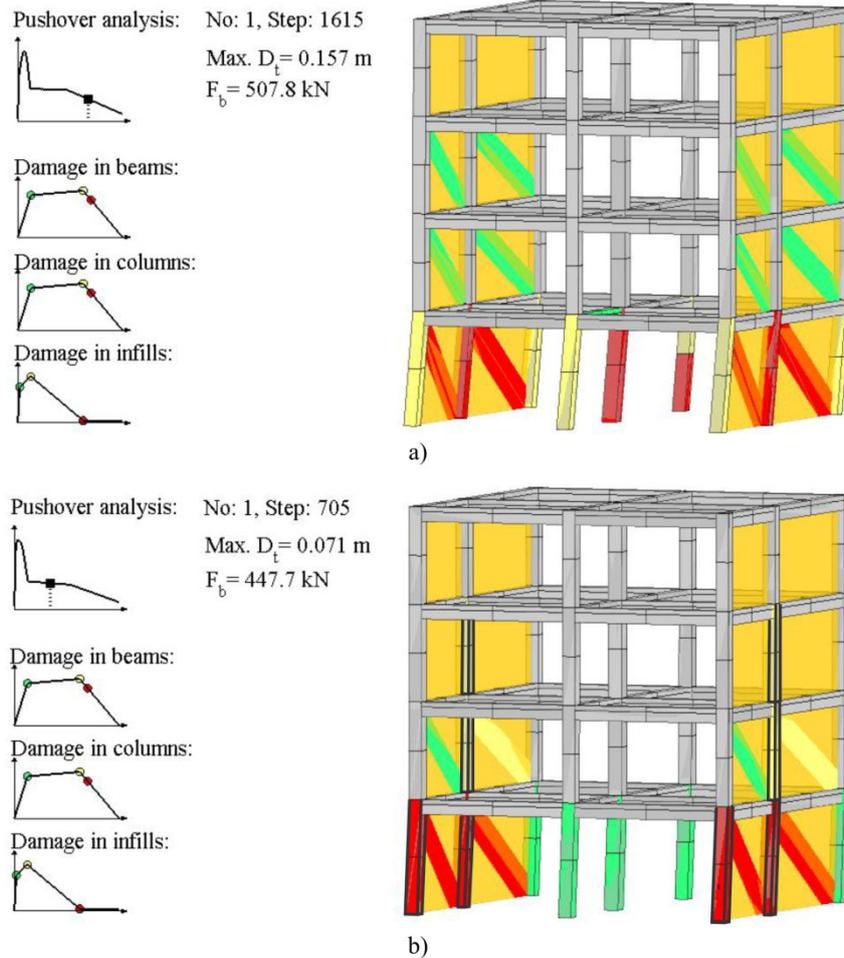
**Figure 4.2.** Pushover curves based on a) pushover analysis (PA) and b) the iterative pushover procedure (IPP), which makes it possible to simulate, approximately, the shear failure of the columns.

The highlighted points on the pushover curves in Figs. 4.2 a) and 4.2 b) indicate the top displacements and the corresponding base shears of the limit state, for which the fragility parameters and the MAF of their violation were assessed. The limit state was defined based on the damage of structural elements in the bottom storey and corresponds to severe structural damage. It was defined that damage limit state is attained when at least 50 % of all structural elements in the first storey had suffered damage corresponded to the near-collapse limit state, as was assumed for the beams and columns.

The top displacements corresponding to the defined limit state (Fig. 4.2) are significantly different if estimated based on the PA or on the IPP. In the case when the shear failures of the columns are neglected (PA), the limit-state top displacement (15.7 cm) is more than twice larger than that (7 cm) corresponding to the case of the IPP, where shear failure of the columns was approximately simulated. The damage for the two cases of analyses is presented in Fig. 4.3. Whereas in the case of PA and IPP all the infills totally collapsed, the damage observed in the columns is significantly different. The flexural failure of columns controls the limit state in the case of PA, whereas in the case of the IPP, some columns fail in shear.

#### 4.2 Fragility analysis and the MAF of limit state exceedance

An equivalent SDOF model was created based on the idealized force-displacement relationship presented in Fig. 4.2. The pushover curve was idealized with a multi-linear force-displacement relationship, defined by five points: the first two points represent the yielding of the idealized system ( $P_1$ ), and the maximum displacement capacity at the end of the plastic plateau ( $P_2$ ), whereas the third point corresponds to the displacement at which the last infill totally collapses ( $P_3$ ), the fourth point corresponds to the displacement at which the strength of the idealized model starts to degrade ( $P_4$ ), and the fifth point is defined by a total loss of the strength ( $P_5$ ). Only a small difference can be observed between the idealized force-displacement relationships and the pushover curves (Fig. 4.2). The mass of the equivalent SDOF models ( $m_{SDOF} = 234$  t), and the modification factor ( $\Gamma = 1.34$ ), which relate the displacement of an equivalent SDOF model to the roof displacement of the MDOF model, were the same for both analysed cases, and were computed as defined elsewhere (e.g. in Fajfar, 2000). The period of the SDOF model amounted to 0.16 and 0.20 s, respectively for the case of PA and the IPP.



**Figure 4.3.** Damage in the structure corresponding to near-collapse limit state for the case of a) PA and b) IPP. The columns, which fail in shear, are marked bold.

The nonlinear dynamic analyses for the equivalent SDOF models were performed by using a set of 30 ground motions (Vamvatsikos and Cornell, 2006), and assuming 5% mass-proportional damping. The output of the nonlinear dynamic analysis for the SDOF models is presented in Fig. 4.4, showing the IDA curves and the points, which link the limit-state top displacement with the peak ground acceleration. These points represent an input for estimating the median limit-state peak ground acceleration  $a_{g50,LS}$  and corresponding logarithmic standard deviation  $\beta_{LS}$ , which are presented in Table 4.1.

The MAFs of limit-state exceedance ( $\lambda_{LS}$ ) were computed according to Eq. (3.1) and are presented in Table 4.1. The hazard function was estimated from the seismic hazard map for the central part of Slovenia ( $k = 3.8$ ). Only the relative difference between the MAFs of limit-state exceedance if estimated based on the result of PA and IPP are discussed in this study. The results indicate that the MAF of exceedance of the defined limit state is about 4 times larger for the case when the shear failure of columns is considered in the analysis (IPP). Thus probabilistic performance assessment of structure may lead to a completely wrong decision regarding structural safety if shear failure of columns due to the effects of masonry infills is neglected. The large difference between the MAFs of limit-state exceedance is also the consequence of the definition of the limit state, which is based on the number of damaged elements in a storey. Clearly, if the shear failure of columns is considered in the analysis, the limit state is attained at a significantly lower top displacement, which significantly contributes to a larger MAF of limit-state exceedance than in the case when the MAF is estimated based on IPP.

**Table 4.1.** Limit-state top displacement, median peak ground acceleration, corresponding dispersion and MAFs of exceeding of the defined limit state based on the PA and IPP.

Quantity	Pushover Analysis (PA)	Iterative Pushover Procedure (IPP)
$D_{t,LS}$ (cm)	15.7	7.00
$\tilde{a}_{g,LS}$ (g)	0.83	0.51
$\beta_{LS}$	0.35	0.26
$\lambda_{LS}$ ( $10^{-4}$ )	0.51	2.17

## 5. CONCLUSIONS

A methodology for the probabilistic seismic performance assessment of reinforced frames with masonry infills has been introduced. The procedure involves simplified nonlinear models, an automated iterative pushover procedure, which enables approximate consideration of the shear failure of columns due to the effects of masonry infills, and nonlinear dynamic analysis of the equivalent SDOF model. It should be noted that the proposed procedure is approximate due to several assumptions, which may have significant impact on the results, such as the determination of the proportion of the force, which is transferred from the infill to the column. On the other hand, it has been shown that the proposed procedure can be used for the assessment of large existing buildings. Another advantage of the proposed procedure is that it goes beyond the limit-state checks of components prescribed in several structural codes, since these checks are, in the proposed procedure, actually embedded in the pushover analysis. The analyst can therefore obtain information on the global response of the structure, which approximately incorporates the effects of failure modes that are usually not simulated in the case of simplified nonlinear models.

The results of the presented example indicate that the shear failure of columns due to the effects of masonry infill may have a significant impact on the seismic response of reinforced concrete frames with masonry infills. Within the presented case study, it was indicated that weak columns cannot resist shear demands induced by masonry infills, which caused a reduction in structural strength. Consequently, the peak ground acceleration that causes violation of a limit state is significantly overestimated if shear failure of the columns is not simulated in the analysis. This was clearly reflected by the mean annual frequency of limit-state exceedance, which was about four times larger than that estimated for the case when shear failure of the columns is neglected.

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