SEISMIC RISK AND LOSS ESTIMATIONS OF REINFORCED CONCRETE FRAME BUILDINGS WITH SIMPLIFIED STRUCTURAL MODEL

A. Jamšek (1), A. Babič (2), M. Dolšek (3)

(1) *Assistant, Dept. of Civil Engineering, University of Ljubljana, ales.jamsek@fgg.uni-lj.si*
(2) *Assistant, Dept. of Civil Engineering, University of Ljubljana, anze.babic@fgg.uni-lj.si*
(3) *Professor, Dept. of Civil Engineering, University of Ljubljana, matjaz.dolsek@fgg.uni-lj.si*

Abstract

The seismic risk and loss estimation of a building portfolio provides key information for optimal decision making about the enhancement of community seismic resilience, but the question arises whether such information is sufficiently reliable, because the seismic performance is currently addressed by fragility functions obtained for a building class, which does not allow damage estimation to a specific building or a ground motion. In this paper, an attempt has been made to address the issue by introducing an improved fish-bone (IFB) model that is computationally non-demanding and robust. Thus, it can be used for structure-specific and ground-motion specific seismic risk and loss estimation related to building portfolio. Firstly, the IFB model, which account for the importance of structural elements of a frame building, is described and its capability is demonstrated by comparing the predicted seismic response of the selected four-story frame building with the results of the pseudo-dynamic tests. It is shown that the simplified structural model was capable of simulating damage at the story level with sufficient accuracy. In the second part of the paper, the IFB model of the four-story reinforced concrete frame building was utilized to estimate the mean annual probability of collapse, the expected annual losses and the annual probability of exceedance of a given loss. It is shown that the results obtained with the IFB models are very similar to those observed by performing seismic analysis with the conventional MDOF model. However, the current version of the IFB model is limited to the simulation of the 2D response.

Keywords: seismic risk assessment; loss estimation; IFB model; MDOF model; reinforced concrete frame buildings

1. Introduction

The evaluation of the seismic response of frame buildings can be based on the multiple-degree-of-freedom (MDOF) models where each beam and column is modelled by at least one finite element. However, to increase the time efficiency and robustness, several simplified MDOF models for seismic analysis of frame buildings, such as different variants of generic-frame (GF) models and fish-bone (FB) models [1-4], have been proposed. Because these models are computationally non-demanding and robust, they have the potential to be especially advantageous in the case of seismic risk assessment of a building portfolio.

In this paper, an attempt is made to utilize a simplified MDOF model of a reinforced concrete frame building for seismic risk and loss estimation. The model is realized as *the improved fish-bone* (IFB) model, which has the same configuration as the initial FB model [1] but contains several improvements. The IFB model can approximately consider different geometry and design of the structural elements, as well as the effective slab widths and the effect of degradation of stiffness. First, the improved fish-bone (IFB) model is briefly described. Then the input data, the configuration of the model and the assumptions used for the definition of the IFB model are presented. Follows the presentation of the capability of the IFB model to simulate the response of pseudo-dynamically tested four-story frame building. In the second part of the paper, the IFB model of the four-story building is used for seismic risk and loss estimation study based on a variant of the PEER methodology [5] where some modifications [6,7] were considered which make it possible to estimate the damage directly from the results of structural analyses. Finally, the capability of the IFB model for fragility analysis and loss estimation is explored by comparing the results obtained by the IFB and conventional MDOF model.

2. IFB model for seismic response analysis of frame buildings

The assumptions related to the IFB model can be divided into two levels. At the first level, the assumptions are the same as those related to the definition of conventional MDOF models of frame buildings [8,9]. Thus, it was assumed that masses are lumped at the story level and that floors are rigid in their planes. The moment-rotation relationship in the plastic hinges of the columns and beams of the IFB model was modelled as bi-linear with the additional linear softening branch [8,9] (Fig. 1). The moment-rotation relationship was therefore defined by three characteristic points (p = 1,2,3) which correspond to three characteristic moments, i.e. the yield moment (M_Y), the maximum moment (M_M) and the ultimate moment (M_U), which are presented in Fig. 1, together with characteristic rotations (Θ_Y, Θ_M, Θ_U). The effective beam widths were determined according to the provisions of Eurocode 2 [10]. The effective stiffness of columns and beams was assumed to be equal to 50 % of the initial stiffness, according to the Eurocode 8 provisions [11].

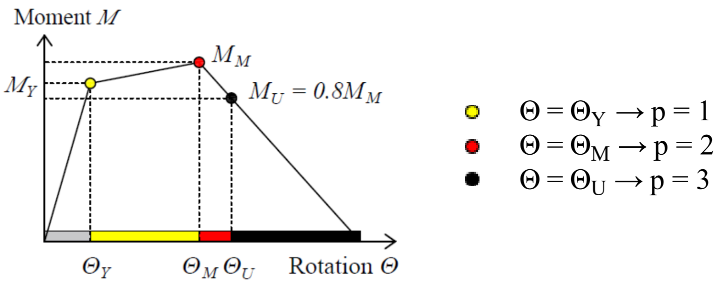

Fig. 1 – Bi-linear moment-rotation relationship with linear post-capping behavior and three characteristic points needed for its definition

The second level assumptions are directly related to the definition of the IFB model. The configuration of structural elements of the IFB model is assumed the same as that of the initial FB model [1]. However, the effective slab widths, the degradation of stiffness and consideration of different geometry and design of the structural elements were also approximately taken into account. Because of the consideration of the effective slab width, the moment-rotation envelopes of the plastic hinges of beams were unsymmetrical, which was not taken into account in the initial FB model. Further on, the height of the IFB columns was assumed to be equal to the building's story height. The moment of inertia of IFB column in a given story was determined by summing the moments of inertia of the columns in that story [2]. Because the length of the beams in a given story can vary significantly, the IFB beam length $L^F_{b,i}$ was defined as one half of the average length of the beams $L_{b,i,k}$ in that story (Fig. 2). One-half of the beam lengths was assumed due to a well-known assumption that in the seismic analysis, the zero moment in a beam is approximately at the middle of its span. Because the IFB model has only two beams per story, the effect of variation of lengths of the beams was accounted for by weighting the moments of inertia of the beams in the given story before summing them up to obtain the moment of inertia of the corresponding IFB beam. The weights for the moments of inertia of the beams were determined as the ratios of the beam lengths to the length of IFB beam. The application of gravity loads on the IFB columns and beams was considered by analogy to the application of gravity loads on the MDOF model, i.e. as point loads on columns and as uniformly distributed loads on beams [9].

The model for the moment-rotation relationship in plastic hinges of IFB columns and beams was the same as that used for the MDOF model (Fig. 1). In general, the properties of the moment-rotation relationship in the IFB column depends on the design approach. Because the investigated building was designed in accordance with the strong column-weak beam approach [11], the characteristic moments of the IFB columns or beams were determined simply by summing the appropriate characteristic moments of all the columns or beams in the story:

$$M^F_{c,i,h,p} = \sum_{j=1}^{m} M_{c,i,j,h,p} \tag{1}$$

$$M^F_{b,i,h,p} = \sum_{k=1}^{n} M_{b,i,k,h,p} \tag{2}$$

where $M^F_{c,i,h,p}$ and $M^F_{b,i,h,p}$ are the p-th characteristic moments of the h-th hinge in the i-th story of the IFB column and beam, respectively. The F denotes variables relevant for the IFB model, b denotes variables related to the beam and c for the column. Moments $M_{c,i,j,h,p}$ and $M_{b,i,k,h,p}$ correspond to characteristic moments of the j-th column and k-th beam of the h-th hinge in the i-th story of the MDOF model.

The characteristic rotations between columns in a given story can vary significantly, which is also the case of the characteristic rotations between beams in a given story. This fact cannot be ignored in the definition of the IFB model. The issues raised can be solved approximately, by defining characteristic rotations of the IFB columns and beams as the weighted average of characteristic rotations and moments of columns and beams:

$$\Theta^F_{c,i,h,p} = \frac{\sum_{j=1}^{m}\left(M_{c,i,j,h,p} \cdot \Theta_{c,i,j,h,p}\right)}{\sum_{j=1}^{m} M_{c,i,j,h,p}} \tag{3}$$

$$\Theta^F_{b,i,h,p} = \frac{\sum_{k=1}^{n}\left(M_{b,i,k,h,p} \cdot \Theta_{b,i,k,h,p}\right)}{\sum_{k=1}^{n} M_{b,i,k,h,p}} \tag{4}$$

where $\Theta^F_{c,i,h,p}$ and $\Theta^F_{b,i,h,p}$ denote the characteristic rotations of the IFB columns and beams, respectively, while $\Theta_{c,i,j,h,p}$ and $\Theta_{b,i,k,h,p}$ are the p-th characteristic rotations of the h-th hinge of the j-th column and k-th beam, respectively, in the i-th story of a frame building. The proposed weights in Eq. (3) and Eq. (4) are the corresponding characteristic moments $M_{c,i,j,h,p}$ and $M_{b,i,k,h,p}$ (Fig. 2).

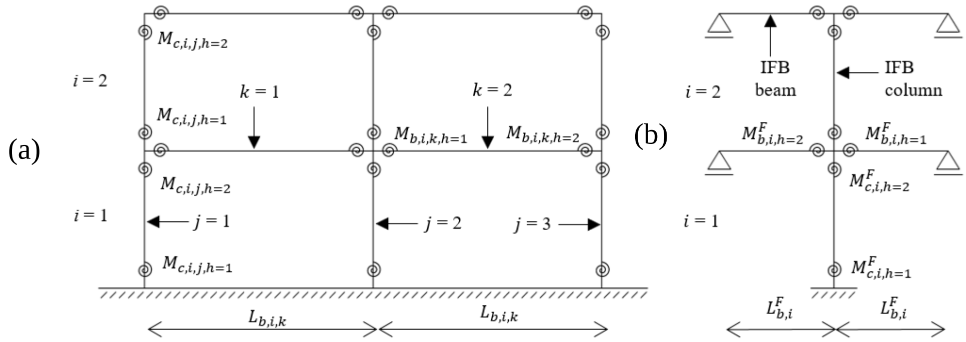

Fig. 2 – Schematic presentation of 2D (a) MDOF and (b) IFB model with the labels of plastic hinges of columns, plastic hinges of beams and beam lengths

The possibility of generating the IFB model was added to the PBEE Toolbox [9], which was developed in Matlab [12] for analysis of MDOF models of frame buildings in OpenSees [13]. As in the case of MDOF models, the moment-rotation relationships were modelled with Hysteretic uniaxial material, where parameter $beta for controlling the unloading stiffness was set to 0.8 [9], while other parameters defining the hysteretic behavior of the plastic hinge were set to 0. Moreover, the P-delta effects were considered, as in the case of the MDOF models.

3. Prediction of engineering demand parameters of the four-story frame building with the IFB model

The capability and accuracy of the IFB model were investigated for the four-story reinforced concrete frame building (Fig. 3), which was pseudo-dynamically tested in full scale at ELSA Laboratory in Ispra, Italy [14, 15]. The building, which is from now on denoted as the PREC8 building, was designed according to the pre-standard of the current Eurocode 8 with the consideration of ductility class high (DCH, behavior factor 5) and PGA = 0.3 g. The building structure was built with concrete C25/30 and Tempcore reinforcement class B500. The story masses amounted to 87 t, 86 t, 83 t and 83 t, from the 1st to the 4th story, respectively. A series of pseudo-dynamic (PsD) tests were performed in Y-direction of the building. In the following, the results of response history analyses are presented for two tests and compared to results of pseudo-dynamic tests. In the first test, the ground motion was scaled to PGA = 0.12 g and in the second test, to PGA = 0.45 g. More information about the building and tests can be found elsewhere [14,15].

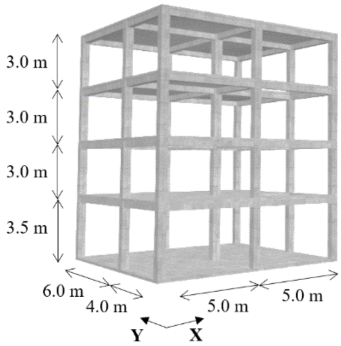

Fig. 3 – The geometry of the pseudo-dynamically tested building PREC8

The seismic response of the four-story building was simulated with the IFB model as presented in Section 2 and with the three-dimensional (3D) MDOF model [9]. In the response history analyses, the viscous damping was disregarded to be consistent with the test assumptions [14].

The results of the IFB model were compared to the response histories of story shears and story drifts observed in the pseudo-dynamic tests, and the response histories of story accelerations, which were calculated from the relative displacement histories and with consideration of ground motions used in the pseudo-dynamic tests. The engineering demand parameters (EDPs) were compared in the 2nd story, where the most extensive damage was observed during the experiments. In Fig. 4, it can be noted that both the IFB and the MDOF model are too flexible for simulating the first low-level demand test (Fig. 4, top row). The models are too flexible because of the consideration of initial stiffness as 50 % of the uncracked stiffness [9]. In this case, the maximum story drift in the 2nd story was overestimated by the IFB model for 174 %. However, the same issue was observed in the case of the MDOF model (Fig. 4b, top row). For the second test, i.e. high-level demand test, both models simulated EDPs quite accurately (Fig. 4, bottom row). The maximum 2nd story shears, drifts and accelerations obtained with the IFB model were overestimated for less than 6 % in comparison with the results obtained with the MDOF model and overestimated for less than 14 % in comparison with the PsD test results. The comparison of results in other stories yielded similar observations.

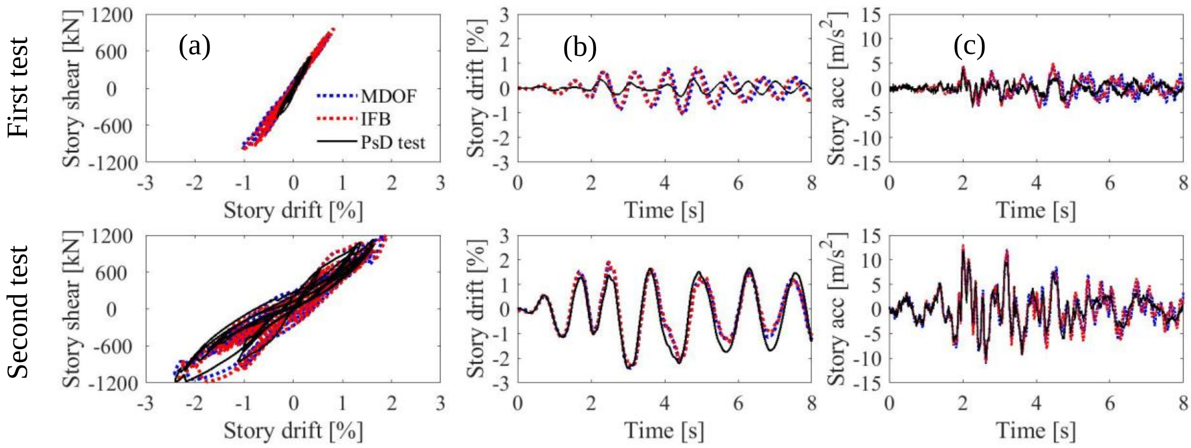

Fig. 4 – (a) 2nd story shear versus 2nd story drift, (b) 2nd story drift history, (c) 2nd story absolute acceleration history for the IFB model and MDOF model of the PREC8 building and the corresponding results of the first and second pseudo-dynamic tests

4. The capability of the IFB model for fragility analysis of the PREC8 building

The presented study aimed to explore further the capability of the IFB model to be used in the fragility analysis. For this purpose, the incremental dynamic analysis (IDA) [16] was performed by using both the IFB and MDOF model of the PREC8 building (Fig. 5). In IDA, a set of 30 hazard-consistent ground motions was considered. The building was assumed to be located in Ljubljana, Slovenia. Therefore, the ground motions were selected by considering the target conditional spectrum [17] defined based on the official seismic hazard maps for the area of Ljubljana and return period of 2475 years [18,19]. The conditioning intensity measure (IM) was set to be the spectral acceleration at the fundamental period of the structure. The 5 % critical damping proportional to the mass matrix was assumed for both models and all the response history analyses presented in this section. The results of IDA showed that the ground motions had to be scaled to very large levels of IM to observe the collapse of the building, which was defined by the dynamic instability of the building model. Therefore, the results of IDA were truncated [20] at the intensity level $S_{ae,max}$ = 3.0 g. The assumption for the selection of the truncation level is explained later in Section 5. Based on the collapse intensities observed in the truncated IDA, the lognormal collapse fragility function was fitted according to the procedure developed by Baker [20]. The IDA results showed a very similar response to the IFB model and MDOF model (Fig. 5a). Consequently, collapse fragility function obtained with the IFB and that obtained with the MDOF model were almost the same (Fig. 5b). Both fragility functions resulted in the probability of collapse in 50 years equal to about 0.09 %. The results of the response history analyses were also the response histories of EDPs (rotations of plastic hinges, inter-story drifts, floor accelerations), which were further on utilized for the damage and loss analyses of structural and non-structural elements.

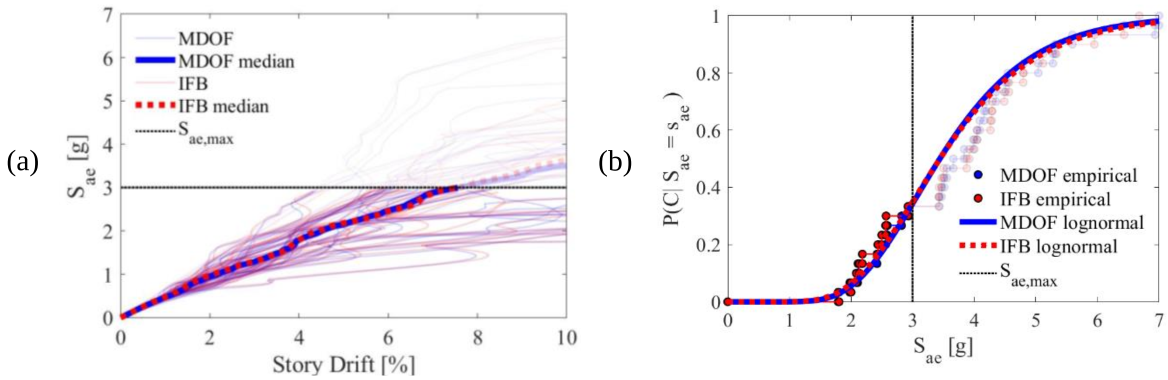

Fig. 5 – (a) The truncated IDA curves for all considered ground motions and the median IDA curves for the IFB and MDOF model, and (b) the sample-based fragility functions and parametrized fragility functions of the IFB and MDOF model based on truncated IDA

5. The capability of the IFB model for loss estimation of the PREC8 building

The results of IDA obtained with the IFB model and MDOF model (Section 4) were further on utilized in the loss estimation of a four-story office building located in Ljubljana for which it was assumed that it has the same structure as the PREC8 building. For an easier representation of the structural and non-structural components, as well as for easier estimation of the quantities of the building components, a 3D BIM model was constructed in ArchiCAD [21] (Fig. 6).

The loss estimation methodology was based on direct simulations of loss and designed as a variant of the PEER methodology [5-7]. Each loss simulation consisted of the response history analysis (Section 4), damage analysis and loss analysis. The results of the loss simulations were further combined with the result of the seismic hazard analysis, which was obtained from previous studies [18,19], to estimate the expected annual loss and the loss curve. In the following, the damage and loss analysis are first described (Sections 5.1 and 5.2, respectively), followed by the results of the loss estimations performed for the IFB and MDOF model.

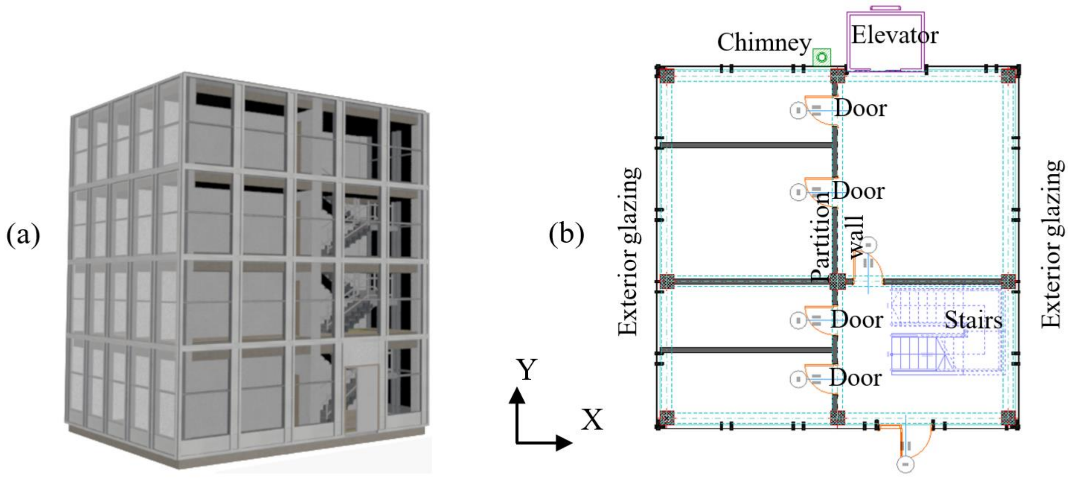

Fig. 6 – (a) The 3D BIM model and (b) a typical story of the building with components considered in damage analysis and loss estimation

5.1 Damage analysis

The goal of the damage analysis was to determine the probabilities of occurrence of the designated damage states at the level of building components based on the EDPs obtained in the response history analysis. The probability of occurrence of damage state of the component j given the value of the corresponding edp $P(DS_j=ds|EDP_j=edp)$ was obtained for each loss simulation as the difference between the probabilities of reaching two sequential damage states (ds = 0, …, m) [22], which were calculated from the component-specific EDP-based fragility functions.

Fragility functions for the non-structural components (Table 1) were obtained from the literature [5,22-24]. Because only 2D response history analyses were conducted, these fragility functions were considered at the story level. However, in the case of structural components (i.e. beams and columns), the fragility functions were defined based on the plastic hinges of the components, which allowed to define a different set of fragility functions for each modelled component according to the actual design of the component. The medians of the fragility functions were defined as the characteristic rotations in the plastic hinges that corresponded to the designated damage states, while the dispersion parameters $\sigma_{\ln,EDP}$ were obtained from the literature [25-27]. For each modelled column and beam, four damage states (DS1–DS4) were defined corresponding to four characteristic rotations, as presented in Fig. 7a and Table 1. Note that values of $\sigma_{\ln,EDP}$ corresponding to DS2 and DS3 were assumed equal to those corresponding to DS1 and DS4, respectively, due to the lack of data.

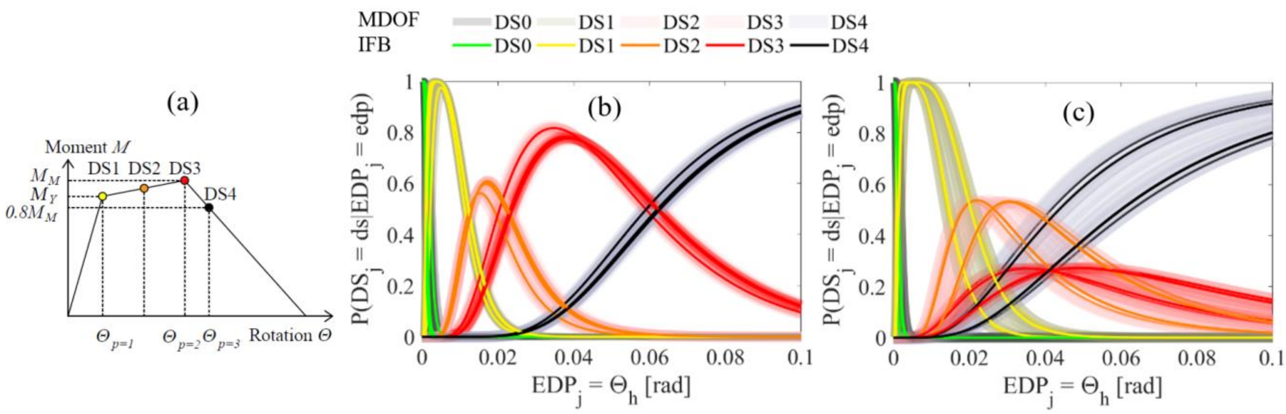

Fig. 7 – (a) The definition of the median rotation causing damage states DS1–DS4 for columns and beams based on the moment-rotation envelopes of plastic hinges, (b) the probabilities of occurrence of damage states DS1–DS4 for columns, and (c) the probabilities of occurrence of damage states DS1–DS4 for beams (the results are presented for the IFB and MDOF model)

In the case of the MDOF model, each structural element was modelled individually and contained two plastic hinges (Fig. 2a). Therefore, in this case, the number of sets of fragility functions was equal to the number of components times two. However, the IFB model consisted of one column and two beams per story. Each column contained two plastic hinges, and each beam contained one plastic hinge (Fig. 2b). Therefore, in this case, the number of sets of fragility functions for columns and beams was equal to the number of stories times two. The lower number of fragility functions defined for the IFB model resulted in a lower number of curves representing the probabilities of occurrence of damage states DS1–DS4 (Fig. 7b and Fig. 7c).

5.2 Loss analysis

The loss analysis was performed by distinguishing the non-collapse and collapse cases [5, 22]. The losses for the non-collapse (NC) cases were estimated based on the damage states observed directly from the results of the response history analysis, as in [6,7]. The loss functions (Table 1), which linked the damage states to the losses, were obtained from the literature or defined based on Slovenian cost databases [28,29]. Only the expected losses corresponding to the damage states were taken into account. The expected loss for components j for a given damage state $E(L_j|DS_j=ds)$ was calculated as:

$$E\left(L_j\middle|DS_j=ds\right)=E'(L_j|DS_j=ds)\cdot q_j\cdot c_{new,j} \tag{5}$$

where $E'(L_j|DS_j=ds)$ is the expected normalized cost of component j, q_j is the quantity of component j and $c_{new,j}$ is the cost of a new component j per unit. The normalized cost $E'(L_j|DS_j=ds)$ was estimated as the ratio of the repair cost of the component $c_{repair,j}$ to the cost of a new component $c_{new,j}$ [6]. The costs of new components $c_{new,j}$ were defined based on estimated average costs in the construction sector in Slovenia in the year 2019 [28,29]. The expected losses of component j $E\left(L_j|EDP=edp\right)$ were then estimated as a function of edp:

$$E\left(L_j\middle|EDP_j=edp\right)=\sum_{ds}E\left(L_j\middle|DS_j=ds\right)\cdot P\left(DS_j=ds|EDP_j=edp\right) \tag{6}$$

The expected losses $E\left(L_j|EDP=edp\right)$ were estimated for each loss simulation. Therefore, when considering a specific level of IM (im) and ground motion (a), the notation for the expected losses of component j was changed into $E(L_{j,a}|EDP_j=edp_a(im))$. Then, the expected losses of all components for the NC case given the level of (im) and ground motion (a) $E\left(L_{T,NC,a}|IM=im\right)$ were calculated by summation of the expected losses corresponding to all components in the building:

$$E\left(L_{T,NC,a}|IM=im\right)=\sum_j E\left(L_{j,a}\middle|EDP_j=edp_a(im)\right) \tag{7}$$

Finally, the total expected losses for a given level of intensity measure (im) and ground motion (a) $E\left(L_{T,a}|IM=im\right)$ were estimated considering three different situations related to the building restoration. The first situation applied, if the collapse of the building was not observed ($IM < im_{C,a}$) and the expected losses for NC case were lower than 40 % of the cost of a new building L_{new}. In such cases, the total losses were equal to $E\left(L_{T,NC,a}|IM=im\right)$. The second situation applied when the collapse of the building was not observed, but the expected losses for the NC case exceeded the threshold of 0.40 L_{new}. If this was the case, the total losses were assumed equal to the replacement cost defined as 1.10 L_{new} [6,29]. The economic threshold of 40 % of the cost of a new building is suggested in many studies because the owners will most likely elect that the building should be replaced if the costs exceed 40% of the costs of a new building [5,30]. The cost of a new building L_{new} was estimated at 476000 € according to Slovenian cost database [29], where the mean value for the mid-priced office and residential buildings amounts to 1100 € per 1.0 m^2 of gross floor area. The third situation applied, if the collapse of the building was observed ($IM \geq im_{C,a}$). In such cases, the total losses were also assumed equal to the replacement cost 1.10 L_{new}. The described model of the total expected losses can be formulated as follows:

$$E\left(L_{T,a}|IM=im\right)=\begin{cases} E\left(L_{T,NC,a}|IM=im\right) & \text{if} \quad IM < im_{C,a} \text{ and } E\left(L_{T,NC,a}|IM=im\right) < 0.4\ L_{new} \\ 1.10\ L_{new} & \text{if} \quad IM < im_{C,a} \text{ and } E\left(L_{T,NC,a}|IM=im\right) \geq 0.4\ L_{new} \\ 1.10\ L_{new} & \text{if} \quad IM \geq im_{C,a} \end{cases} \tag{8}$$

Table 1 – Parameters of fragility functions and loss functions ($\widetilde{EDP}$ – median EDP causing the damage state, $\sigma_{ln,EDP}$ – the logarithmic standard deviation of EDP values causing the damage state, $c_{new,j}$ – cost of a new component j, $E'(L_j|DS_j=ds)$ – normalized losses due to damage state ds)

Component	**Description**	**Fragility functions**				**Loss functions**	
		DS_j	**EDP_j**	**$\widetilde{EDP}_j$**	**$\sigma_{ln,EDP}$**	**$c_{new,j}$ [€/unit]**	**$E'(L_j\|DS_j=ds)$**
Column	Reinforced concrete column	DS1	Θ [rad]	$\Theta_{p=1}$	0.36	900 €/unit	0.17
		DS2		$(\Theta_{p=1}+\Theta_{p=2})/2$	0.36		0.83
		DS3		$\Theta_{p=2}$	0.40		1.00
		DS4		$\Theta_{p=3}$	0.40		1.00
Beam	Reinforced concrete beam with effective slab widths	DS1	Θ [rad]	$\Theta_{p=1}$	0.36	1590 €/unit	0.18
		DS2		$(\Theta_{p=1}+\Theta_{p=2})/2$	0.36		0.81
		DS3		$\Theta_{p=2}$	0.60		1.00
		DS4		$\Theta_{p=3}$	0.60		1.00
Partition wall	Gypsum – board partitions	DS1	IDR [%]	0.34	0.56	58 €/m²	0.10
		DS2		0.78	0.27		0.60
		DS3		1.10	0.25		1.20
Exterior glazing	Exterior glazing – horizontal wall system	DS1	IDR [%]	4.00	0.36	840 €/unit (1.5 m x 1.8 m)	1.00
		DS2		4.60	0.33		1.00
Chimney	Masonry chimney	DS1	PFA [g]	0.30	0.60	150 €/m	0.87
		DS2		0.50	0.60		1.20
Roof	Clay tile roof	DS1	PFA [g]	1.50	0.40	70 €/m²	0.41
		DS2		1.90	0.40		1.00
Suspended ceiling	Ceiling systems - suspended acoustical type	DS1	PFA [g]	0.22	0.40	24 €/m²	0.12
		DS2		0.65	0.50		0.36
		DS3		1.23	0.55		1.20
Stair	Reinforced concrete stairs cast in place	DS1	IDR [%]	0.50	0.60	1780 €/unit	0.223
		DS2		1.70	0.60		0.45
		DS3		2.80	0.45		1.00
Door	Interior glass door	DS1	IDR [%]	3.03	0.50	450 €/unit	0.60
		DS2		4.13	0.50		1.00
		DS3		5.10	0.30		1.00
Elevator	Elevator - semi-automatic glass door	DS1	PGA [g]	0.34	0.28	18000 €/unit	1.00
Generic drift sensitive	Vertical piping, bath tubs, fire hose cabinet	DS1	IDR [%]	0.34	0.50	13000 €/story	0.025
		DS2		0.80	0.50		0.10
		DS3		2.50	0.50		0.60
		DS4		5.00	0.50		1.20
Generic acceleration sensitive	Plumbing, toilets, HVAC, heating, cooling	DS1	PFA [g]	0.21	0.60	13000 €/story	0.020
		DS2		0.50	0.60		0.12
		DS3		1.00	0.60		0.36
		DS4		2.00	0.60		1.20

Table 2 – The quantities of components q_j per story, the cost in a typical story and cost of new components $\Sigma c_{new,j}$ for the whole building

Component	Unit	Story	Quantity q_j/ story	Cost of new component $\Sigma c_{new,j}$	
				Typical story	Whole building
Column	/ unit	1. - 4.	9	8100 €	33300 €
Beam	/ unit	1. - 4.	6	9540 €	38200 €
Partition wall	/ m²	1. - 4.	23.4	1360 €	5400 €
Exterior glazing	/ panel	1. - 4.	20	16800 €	67200 €
Chimney	/ m	1.	14	2100 €	2100 €
Roof	/ m²	4.	108.2	7570 €	7600 €
Suspended ceiling	/ m²	1. - 4.	90.8	2180 €	8700 €
Stair	/ unit	1. - 3.	1	1780 €	5300 €
Door	/ unit	1. - 4.	4	1800 €	7200 €
Elevator	/ unit	1. - 4.	1	18000 €	18000 €
Generic drift sensitive	/ floor	1. - 4.	1	13000 €	52000 €
Generic acceleration sensitive	/ floor	1. - 4.	1	13000 €	52000 €

The data for defining the expected losses of components is presented in Table 1 [5,28,29]. The cost of new generic components that are characterized as drift and acceleration sensitive (Table 1) were estimated based on a description from the literature [22,23] and with consideration of Slovenian cost databases [28,29]. For the structural components, the normalized cost $E'(L_j|DS_j=ds)$ was estimated according to the proposed reasonable repairs and corresponding costs [28] for damage levels observed during cyclic tests performed on reinforced concrete columns [31] (Table 1). For the component "Stair" the normalized cost $E'(L_j|DS_j=ds)$ was estimated by considering the damage states of stairs as presented in FEMA PACT tool [5] and the costs for the repair of such damage based on Slovenian construction and design practice [28,29] (Table 1). The costs of new components were also estimated based on the average prices in the Slovenian construction and design practice [28,29]. The quantities of components and the costs of new components in a typical story and the whole building are presented in Table 2.

5.3 The results of loss estimation

Loss estimation was performed for the IFB and MDOF model. In the following, a comparison is made between the results obtained with the two models, where the results are expressed by various performance measures. In the case of both models, the losses estimated for earthquakes with the return period of less or equal to 10 years were neglected, due to the assumption that such frequent earthquakes cause only negligible damage. This assumption is also the consequence of the overestimated demands in the case of low levels of IM, as shown in Fig. 4 (top row).

The expected cumulative losses corresponding to the structural components, non-structural drift (IDR) sensitive components and non-structural acceleration (PFA) sensitive components are presented in Fig. 8 for the non-collapse cases and different levels of IM. It can be observed that the losses estimated with the IFB model are very similar to those estimated with the MDOF model. Additionally, the losses of non-structural PFA sensitive components contribute the most in the case of low levels of IM, while for higher levels of IM, the contribution of all three types of components is similar (Fig. 8a).

The total expected losses at different levels of IM, which comprise losses from both the non-collapse and collapse cases with the consideration of different situations regarding the restoration of the building (Eq. (8)) are presented in Fig. 8b, while the probabilities of exceeding selected thresholds of total loss at different levels of IM are presented in Fig. 8c. In both cases, the matching between the results obtained with the IFB model and the results obtained with the MDOF model is very good. The results presented in Fig. 8b and Fig. 8c also indicate that simulations with S_{ae} higher than 3.0 g did not affect the results of the loss estimation, because the probability of the loss being equal to the replacement costs is equal to 1.0 for the S_{ae} larger than 3.0 g. This observation justifies the selection of the truncation level in the IDA analysis ($S_{ae,max}$ = 3.0 g; Section 4). The insignificance of S_{ae} levels above 3.0 g is a consequence of the assumption that the owners will most likely select the reconstruction over the repair of the damaged building if the repair exceeds 40 % of the cost of a new building. This threshold was exceeded for all the selected ground motions at a level of S_{ae} below 3.0 g.

The accuracy of the IFB model was also confirmed by comparing the loss curves obtained with the MDOF model (Fig. 8d). The curves are presented in the form of the mean annual frequencies of exceeding a designated value of total loss $\lambda(L_t > l_t)$ due to the earthquakes (Fig. 8d). For loss curves of both models, a plateau can be observed for values higher than 0.40 L_{new}, which is, again, a consequence of the assumption that the replacement of the damaged building will be elected in cases of repair costs above 0.40 L_{new}.

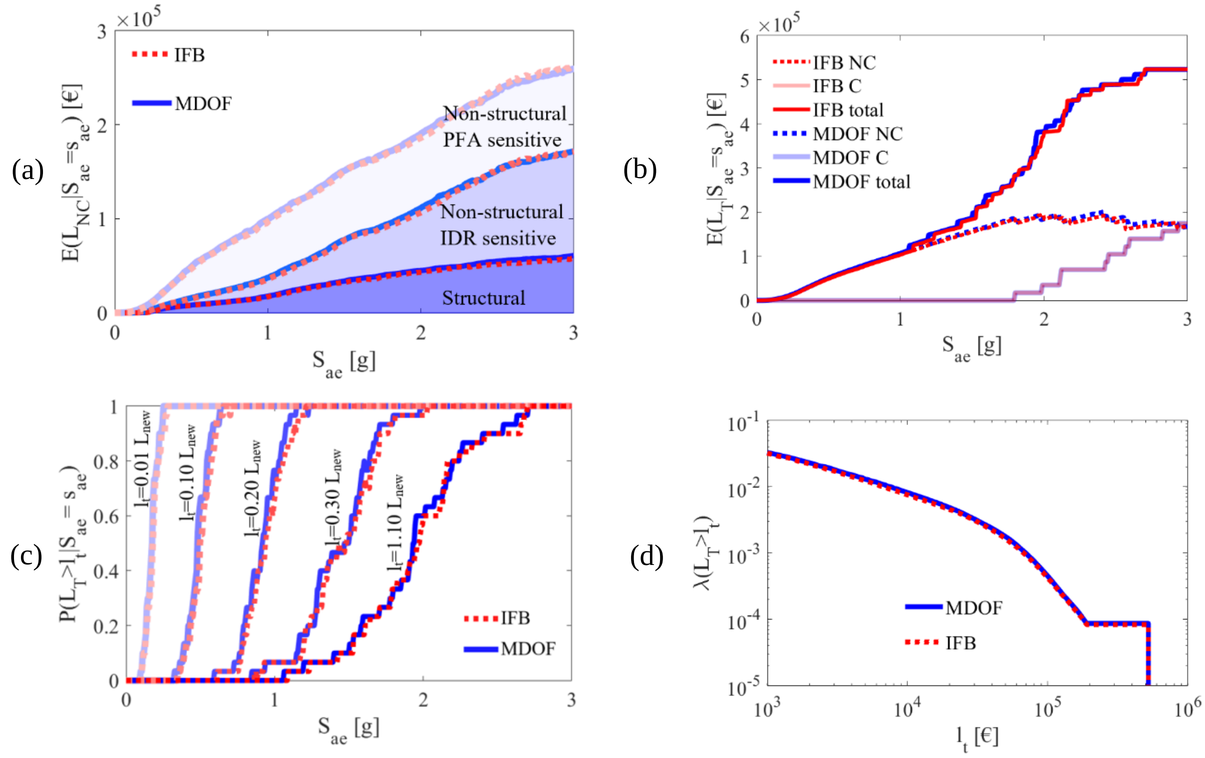

Fig. 8 – (a) The disaggregation of the mean estimated cumulative losses for non-collapse cases as a function of IM, (b) the expected total losses given the level of IM with a breakdown to the corresponding losses due to collapse and non-collapse cases, (c) the probability of exceeding a designated value of loss given the level of IM, and (d) the loss curve (the results are presented for the IFB and MDOF model)

Finally, the expected annual losses EAL were also calculated. They amounted to 369 € and 365 € for the IFB and MDOF model, respectively, which approximately corresponded to 0.078 % of the cost of a new building L_{new} or 85 € per 100 m^2 of gross floor area. The error in the EAL when utilizing the IFB model is only 1 % comparing to the EAL estimated with the MDOF model.

6. Conclusions

The improved fish-bone (IFB) model, which is a simplified MDOF model that can be used for simulation of the response of predominantly plan-symmetrical frame buildings, was presented. The configuration of structural elements of the model is the same as that of the initial fish-bone model, which has been introduced in previous study [1]. However, the proposed model contains several novelties regarding the estimation of the properties of the structural elements of the model. These novelties can improve prediction of seismic performance of a building, which was demonstrated for a four-story reinforced concrete frame building. In this case, the IFB model simulated the story drifts, story shears and story accelerations almost as accurate as the MDOF model.

Further on, the IFB model was utilized for fragility analysis and loss estimation of a four-story reinforced concrete frame building. The damage of the building components and losses were estimated directly from seismic demands observed in the response history analyses. The results showed that the probability of collapse, loss curves and expected annual losses estimated by using the IFB model were comparable to the results based on the MDOF model. The results are very promising, although the IFB model is simplified. However, additional studies are needed in order to better understand pros and cons of the IFB model. Because current version of the IFB model is capable of simulating only a 2D response of frame buildings, it is necessary to investigate possibilities of extending the IFB model to 3D buildings without reducing computational efficiency and robustness. It is foreseen that new developments could make it possible seismic risk and loss estimations of building portfolios by performing nonlinear response histories of specific structures of building portfolios.

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