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Improved fish-bone model: a simplified structural model for seismic analysis of older and contemporary reinforced concrete frames

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Abstract

A new simplified structural model, which is termed as the improved fish-bone model (IFB), is presented in the paper's first part. The model is an extension of the conventional fish-bone model. It can be used for seismic analysis of older and contemporary predominantly plan-symmetric reinforced concrete frame buildings. The IFB model approximately accounts for the importance of structural elements and the potential redistribution of demands between frame buildings' elements. The latter is crucial for the seismic analysis of older frame buildings. Because of reduced degrees of freedom, it can be easily used to perform numerous numerical simulations based on non-linear response history analysis, demonstrated in the second part of the paper. The IFB model capability and accuracy is shown for the most important engineering demand parameters of selected pseudo-dynamically tested full-scale frame buildings. Further on, the IFB model's is used to estimate the fragility functions based on the response history analyses for a set of selected accelerograms. It is shown that the new IFB model is capable to accurately simulate the non-linear seismic response of a contemporary and older frame building compared to the simulated response of the conventional MDOF model. The main advantages of the IFB model are the computational robustness and computational efficiency, which becomes particularly pronounced for the buildings with many structural elements and seismic loss estimations of buildings and the response history analyses of building portfolios.

Key words: improved fish-bone model, MDOF model, reinforced concrete frame, pseudo-dynamic test, seismic analysis, computational time efficiency

1 Introduction

Seismic response of building structures is often evaluated by multi-degree-of-freedom (MDOF) models (Figure 1a), where one finite element model is used for each column, beam or wall in a storey. However, many simplified MDOF models [1-5] have been developed, where their main advantages are the computational efficiency and robustness, while the accuracy of the models is not excessively reduced. A significant number of models were developed as a variant of the generic frame (GF) (Figure 1c) and fish-bone (FB) models (Figure 1b) to study the response of frame buildings [1-5]. The GF model proposed by Nakashima et al. [1] was used for estimating the seismic demand of steel moment-resisting frames [2]. In contrast, the initial idea of the fish-bone model was presented by Ogawa et al. [3]. Later on, Khaloo and Khosravi [4] modified the fish-bone model to more accurately simulate storey drift demands of analysed moment-resisting frames. One of the last developments was the development of the improved fish-bone (IFB) model, which can predict the seismic response of older and contemporary reinforced concrete frame buildings with sufficient accuracy [5, 6].

Although simplified models are computationally efficient, their capacity for simulating phenomena observed during the seismic response of buildings is still not yet entirely understood. Although the detailed MDOF models can provide a reliable estimation of seismic demands in structural elements of building structures, there are several potential applications where analysts cannot afford to use the detailed MDOF models. One example is the seismic risk assessment and loss estimations of a building portfolio. The building models of building portfolio have to be simplified either due to the lack of data or limited resources available for seismic risk estimation [7].

In the first part of the paper, the IFB model [5] is briefly presented. It is developed as a variant of the fish-bone (FB) model (Figure 1b) [3]. The basic configuration of elements of the presented IFB model is equivalent to that of the FB model. The main novelty is related to the estimation of the parameters of the beams and columns of the IFB model. The latter is especially important if beams are stronger than columns, which can be the case for older frame buildings, or if columns or beams in one storey of a frame building differ significantly from each other. In the second part of the paper, the IFB model capabilities are demonstrated by means of the seismic response of two buildings, a contemporary four-storey frame building and an older four-storey frame building. Both buildings were tested in full scale [8, 9]. The IFB models are then used to predict IDA curves and fragility functions analyses for a set of selected ground motions. Additional capabilities of the IFB model, such as pushover and damage analyses, are demonstrated in previous studies [5, 6].

2 A description of improved fish-bone model

Although several simplified MDOF models have been developed, the new IFB model's development was mostly based on the FB model's configuration [3]. The FB model consists of a single column and a pair of beams per each storey. The beams of the fish-bone model are half the beams' length and are restrained at one end to prevent vertical displacements (Figure 1b) [3, 4]. The FB model was further simplified into a generic frame (GF) model [1], where all the beams are condensed into a single rotational spring. For both the FB and GF model, the point of contra-flexure of all beams is assumed at the middle of the span. The masses are assumed lumped at floor levels, and plastic hinges model material nonlinearity at member ends. The GF model's main disadvantage is that the P-delta effects can only be considered as a first-order approximation where the storey shear resistance is subtracted by the effect of the axial force and storey displacements [1]. Also, the use of the simplified models is limited, where one of the main limitations studied by Nakashima et al. [1] was the neglection of the effect of the column and beam elongation and contraction. The same authors concluded that the prediction of seismic response obtained with the simplified model is not significantly affected if frames have more than 1-bay or the frame height is less than three times its length. All the frames analysed in this study fulfil these conditions. The second considerable limitation is that the presented model is limited to analyses of predominantly plan-symmetrical frame buildings [5, 6].

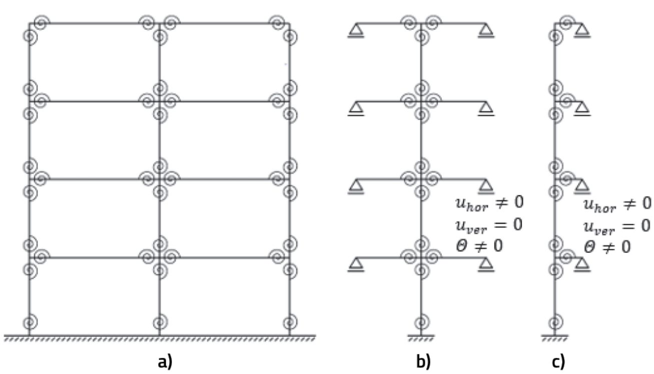

Figure 1. Example of 2D: a) MDOF model of a frame, b) equivalent fish-bone (FB) model, c) equivalent generic-frame (GF) model

2.1 Description of the IFB model

For the definition of the IFB model, the building's information can be the same as that required for the definition of MDOF model. In the optimal case, the building's required information is the building geometry in plan and elevation, reinforcement drawings, material characteristics of designed or built-in material, the storey masses, and the gravity loads. For the IFB model's definition, some steps are the same as for the definition of the input data of the MDOF model. This means that the geometrical constants of the structural elements (e.g. length L, cross-section area A, the moment of inertia I), gravity loads and calculate the corresponding loads on the beams and columns need to be defined. Then, the moment-rotation relationship in plastic hinges of columns and beams has to be estimated.

When defining the IFB model of a building, there are two levels of assumptions. The first level of assumptions is already used for MDOF models [10], which are considered sufficiently accurate to simulate the seismic response in the range up to the near-collapse [10]. It is usually assumed that the mass can be modelled by lumped storey mass in the centre of gravity and that floor is rigid in its plane. The strain-stress relationship for the moment-curvature analysis of the columns and beams' cross-section can be calculated according to the code requirements (e.g. [11]). It was assumed that the moment-rotation relationship in the plastic hinges is modelled by a bi-linear relationship with consideration of the linear softening branch in the post-capping range (see Figure 2). Three characteristic points thus define the moment-rotation relationship. They correspond to yield (M_Y), maximum moment (M_M) and ultimate moment (M_U), which are from now on called characteristic moments, while the corresponding rotations are termed as characteristic rotation (Θ_Y, Θ_M, Θ_U) (Figure 2). Note that the gravity analysis's axial forces were considered for the calculation of yield and maximum moments. It was assumed that the variation of axial forces does not significantly impact the seismic response of columns. The axial force in the beams was considered zero. Additionally, the effective beam widths were assumed according to Eurocode 2 [11] and the columns' and beams' effective stiffness as 50 % of the initial stiffness [12].

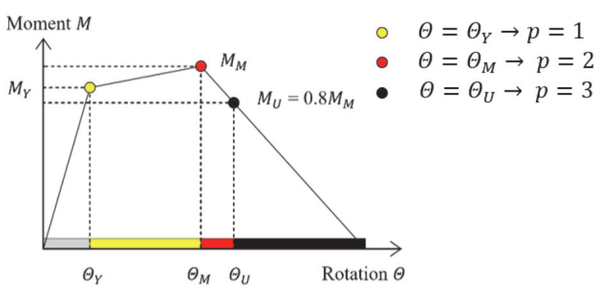

Figure 2. Bi-linear moment-rotation relationship with linear post-capping behaviour and three characteristic points needed for its definition.

The second level of assumptions refers to the definition of the IFB model. These assumptions are related to the procedures used to condense the input data of the MDOF model to the input data of the IFB model. In order to distinguish between the structural elements of the MDOF model and the IFB model, the column and beam of the IFB model are from now on denoted as IFB column and IFB beam, respectively. In the definition of the IFB model, it is assumed that the length of IFB columns is equal to the storey height.

Simultaneously, the corresponding moment inertia is equal to the sum of moments of inertia of columns in the corresponding storey of a frame [1]. For the IFB beams' length, it is assumed that it is equal to one half of the beams' average length in a storey [5]. Half is used because the IFB model's configuration is such that the beams are restrained at the mid-span, the same as proposed for the FB model [3, 4]. The vertical loads in the IFB model can be applied in the same manner as for the MDOF model [10], which means that applying point loads in joint IFB columns and beams and distributed line loads on the IFB beams [5].

The most important feature for estimating the seismic response of frame buildings with the new IFB model is the definition of the moment-rotation relationship of the plastic hinges of the IFB columns and beams, which is, however, also a subject of several simplifications. For this study, the moment-rotation relationship in plastic hinges of the IFB columns and beams is assumed bi-linear with an additional linear post-capping behaviour as presented in Figure 2. The -th characteristic moment of the *h*-th plastic hinge of the *i*-th storey column $M^F_{c,i,h,p}$ is defined as the sum of the characteristic moments of plastic hinges of the columns:

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

where $M_{c,i,h,p}$ in Eq. (1) is the *p*-th characteristic moment of the *h*-th plastic hinge of the *j*-th column of the *i*-th storey of a frame building. The estimation of the above-introduced parameters of the model is straightforward. On the other hand, more attention was given to estimating the moment-rotation relationship of the IFB beams to improve the simulation of seismic response of frames. The latter was proven to be especially important for older frames, which were not designed considering the strong column – weak beam concept (e.g. without consideration of capacity design according to Eurocode 8 provisions [12]). In this case, a new procedure was defined, so that the reduced contribution of the beam strength is considered if the beams in a joint are stronger than corresponding columns. On the other hand, if columns in a joint are stronger than beams, the full contribution of beam strength can be considered. Therefore, the reduced or full contribution of beam strength $M^\Delta_{b,i.k.h,p}$ first needs to be defined for *h*-th hinge, *k*-th beam in the *I*-th storey of the analysed frame building [5]. The *p*-th characteristic moment of the *h*-th hinge in the *i* -th storey $M^F_{b,i.h,p}$ is then estimated as the sum of corresponding characteristic moments:

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

More details are presented elsewhere [5, 6]. For a more straightforward presentation of indices, see Figure 3.

The characteristic rotations in plastic hinges of the IFB beams and IFB columns should be carefully defined [5] in order to define the model, which is capable of simulating the most important failure modes of the frames. It was observed that it is essential to consider the variation of the characteristic moments and rotations from column-to-column and beam-to-beam. Therefore, it is was proposed that the characteristic rotations of the plastic hinge of the IFB model are defined as the weighted average of the characteristic rotations of the corresponding plastic hinges of the MDOF model [5]. The weighted average of characteristic rotations of the IFB beam and column is proposed because the characteristic moments in plastic hinges of the columns and beams (M_Y, M_M, M_U) correspond to different values of the corresponding characteristic rotations. The latter cannot be directly accounted for by defining the IFB beam and column's moment-rotation envelopes. With the proposed weights, the elements with greater strength significantly influence the characteristic rotations in the IFB columns and beams.

The proposed IFB model was investigated by OpenSees [13]. Hysteretic behaviour in plastic hinges was modelled by a uniaxial material *Hysteretic*, a typical peak-oriented model, with hysteretic rules similar to Takeda's. The presented IFB model was incorporated and analysed with Performance-based Earthquake Engineering (PBEE) Toolbox presented by Dolšek [14], which is based on OpenSees [13] and Matlab [15].

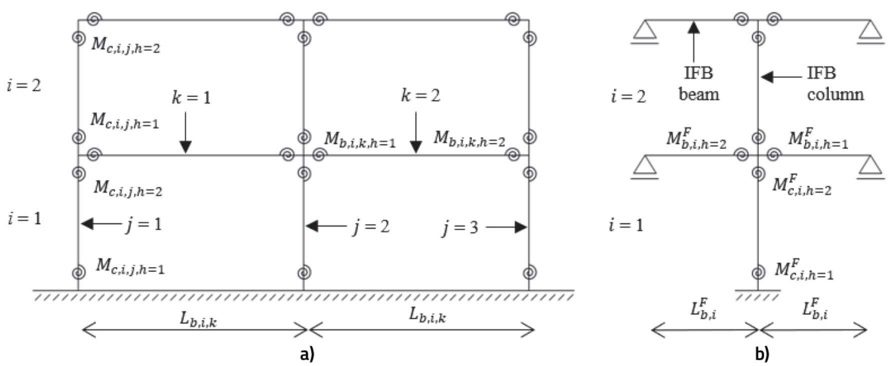

Figure 3. Presentation of indices for plastic hinges of columns and beams of (a) MDOF and (b) IFB model

3 Description of the analysed buildings

A 4-storey frame building [8] denoted PREC8 building (Figure 4a) and 4-storey frame [9] indicated ICONS building (Figure 4b) were used to demonstrate the capability of the IFB model. Both buildings were pseudo-dynamically tested in full-scale at ELSA Laboratory. The PREC8 building is a 4-storey reinforced concrete frame building (Figure 4a) designed following the pre-standard of the current Eurocode 8 by assuming ductility class high (DCH, behaviour factor 5), the medium soil condition and the design PGA equal to 0.3 g. The building was constructed with concrete class C25/30 and Tempcore reinforcement steel class B500. PREC8 building was tested by a series of pseudo-dynamic tests in Y-direction [8]. In the definition of the building models of the investigated structure, the concrete strength was assumed to be equal to 42 MPa, while the mean yield strength differed depending on the diameter of the reinforcing bar, while the average value amounted to approximately 580 MPa. Storey masses from the 1st to the 4th storey were equal to 87 t, 86 t, 83 t and 83 t, respectively [14]. The second building analysed is the ICONS building (Figure 4b) designed to gravity loads only. The building was also pseudo-dynamically tested at ELSA Laboratory in Ispra [9]. The concrete strength of 16 MPa and the reinforcement steel tensile strength of longitudinal reinforcement of 343 MPa were assumed in the analysis. For the model's definition the masses for 1st, 2nd and 3rd storeys amounted to 46 t, while the mass of the 4th storey was considered equal to 40 t. The pseudo-dynamic test was terminated prematurely because of severe damage observed at the top of the "strong" column in the 3rd storey, which induced a soft storey mechanism [9].

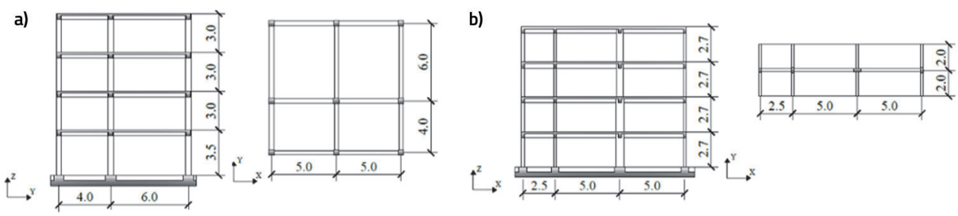

Figure 4. The elevation and plan views of the: a) PREC8; b) ICONS building

The IFB models of pseudo-dynamically tested buildings were validated for several engineering demands parameters (i.e. storey drifts, storey accelerations and storey accelerations) estimated with the detailed MDOF model and the pseudo-dynamic (PsD) test results [5, 6]. Storey shear versus storey drift response history of pseudo-dynamically tested buildings is presented in Figure 5. The results are given for the most damaged storey of the buildings. It can be observed that the IFB model simulations are quite similar to that of detailed MDOF model and the experimental results. The IFB model slightly underestimated the maximum drift in the 3rd storey of ICONS building. However, the difference is not relevant to risk studies because ICONS building is susceptible to the variation of ground-motion intensity for the level of ground motions considered in the pseudo-dynamic test. More results are presented elsewhere (i.e. [5, 6]).

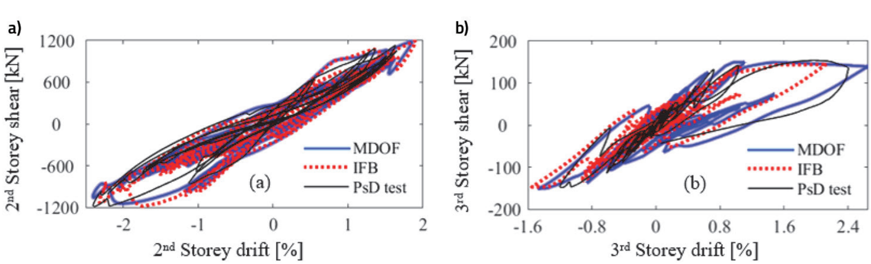

Figure 5. Storey shear versus storey drift for: a) 2nd storey of PREC8 building; b) 3rd storey of ICONS building

4 Capability of the IFB model for the incremental dynamic analysis and fragility analysis

The incremental dynamic analysis (IDA) and fragility analysis were performed for buildings described in Section 3 (Figure 4). The IFB model's accuracy was estimated by relative error, i.e. by comparing the results of the seismic response of the IFB model to those estimated by the MDOF model. Please note that all the MDOF models were defined based on previous studies [14, 16].

The capabilities of the IFB model was verified for three limit states. The LS1, LS2 and LS3 were defined considering different levels of damage in the frame building. For the IFB model, it was assumed that the limit state is attained when the rotation in one of the plastic hinges of the IFB columns exceeds the limit-state characteristic rotations from the moment-rotation relationship of the IFB columns. In the case of the MDOF model, the limit-states were defined at the level of a storey to make the limit state definitions of the two structural models consistent. It is thus assumed that LS1, LS2 or LS3 of the MDOF model are attained when the weighted average rotation demand of the lower (h = 1) or upper (h = 2) plastic hinges of columns exceed the weighted average characteristic rotations of the moment-rotation relationship of the corresponding plastic hinges [5].

The incremental dynamic analyses (IDA) [17] were performed, taking into account a set of 30 hazard-consistent ground motions (GM) which are presented elsewhere [5]. The IDA curves for maximum storey drift obtained by utilising the IFB and MDOF models are very similar even if they are compared for a given ground motion (Figure 6). Consequently, also the median IDA curves of the IFB and MDOF models match very well for PREC8 and ICONS building. The points in the IDA curves indicate the attainment of limit states LS1, LS2 and LS3 and define the limit-state spectral acceleration at the fundamental period of a building ($S_{ae,LS}$) (Figure 6). With the IFB models the limit-state spectral accelerations $S_{ae,LS}$ are quite accurately estimated compared to those obtained by the MDOF model. However, a slightly more significant difference can be observed in the estimation of limit-state maximum storey drifts. However, this difference is not so relevant because a small increment in the spectral acceleration results in a considerable increment in the storey drift demands, especially in the near-collapse range.

The fragility functions were then defined in the form of lognormal cumulative distribution functions (Figure 6). Therefore, it was necessary to estimate the median value of the limit-state spectral acceleration at the fundamental period of the building model $\tilde{S}_{ae,LS}$ and the standard deviation of logarithmic values of limit-state spectral acceleration at the fundamental period of the building β_{LS} (e.g. [18]). Note that the fragility functions are also presented in the form of the empirical cumulative distribution functions obtained directly from the limit-state spectral accelerations resulting from the response history analysis either using IFB or MDOF model (Figure 5). The median values of limit-state spectral accelerations are presented for IFB and MDOF model of both analysed buildings in Table 1. It can be observed that the highest relative error amounts to 13 % for the LS1 for PREC8 building, due to the small intensity measure level, while, for the LS2 and LS3 relative error is less than 2 %. On the other hand, for ICONS building, the relative errors amounted to less than 2 % for all limit-states.

Table 1. The median value of limit-state spectral accelerations $\tilde{S}_{ae,LS}$ estimated by using the IFB and MDOF models of the PREC8 and ICONS building and the corresponding relative error.

	Limit state	IFB	MDOF	Relative error
PREC8	LS1	0.48	0.42	+13 %
	LS2	1.55	1.56	-0 %
	LS3	2.79	2.83	-2 %
ICONS	LS1	0.13	0.13	+2 %
	LS2	0.52	0.52	-1 %
	LS3	0.65	0.64	+2 %

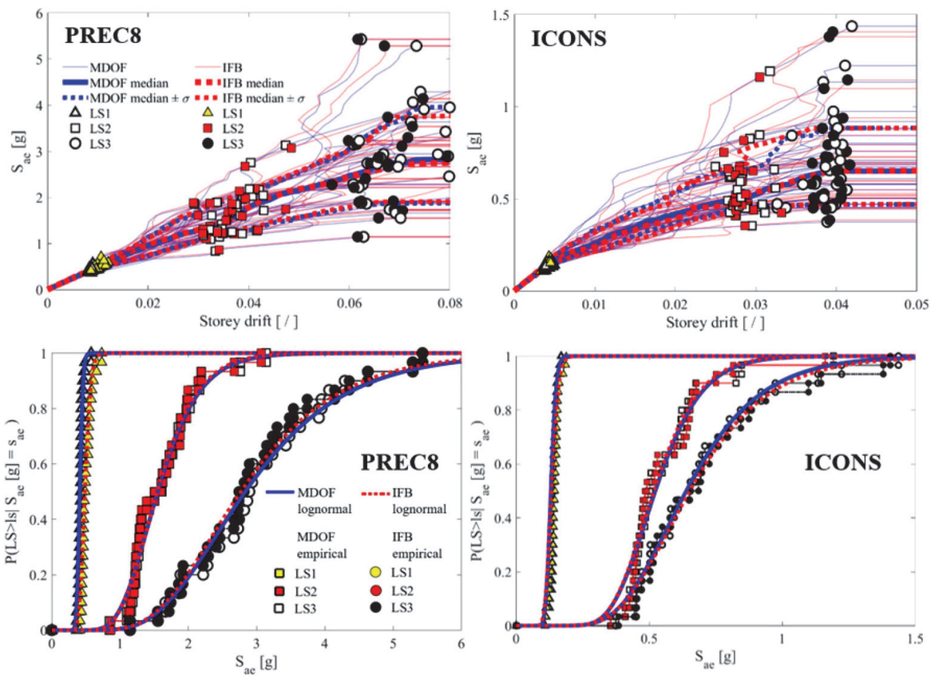

Figure 6. IDA curves of each considered ground motion, the median, 16th and 84th percentile of IDA curves and points of the attained limit states. The empirical and lognormal fragility functions for the IFB and MDOF models of the PREC8 and ICONS building

In addition to the IFB model's accuracy presented for both analysed buildings, the computational efficiency and robustness of numerical analyses are also important for carrying out the seismic fragility analysis, especially if the objective is a seismic risk assessment of building portfolio. The computational time required for performing analyses by utilising IFB models is several times shorter to the time needed for carrying out the analyses by the conventional MDOF models, but for realistic buildings, where the number of elements is significantly higher, the computational time in the case of IFB models is only a fraction of that required to perform non-linear response history analysis of MDOF models [5, 6].

5 Conclusions

The IFB model was briefly presented, and its capability was demonstrated utilizing the seismic analyses of four-storey contemporary and four-storey older reinforced concrete frame building, which were pseudo-dynamically tested. The results of the simulation of the seismic response the selected building structures proved that the IFB model provides results which are practically as accurate as those of the conventional MDOF model when applied to predominantly plan-symmetric frame buildings. It was shown that the IFB model could be used to estimate the capacity of the examined buildings by IDA and fragility analysis. The overall maximum relative error estimated for the median limit-state spectral accelerations obtained by IDA was observed less than 13 %, while an average overall error was observed only about 3 %. The presented IFB model's main limitation is that its use is currently limited to analyses of predominantly plan-symmetrical frame buildings. Also, the IFB model cannot simulate all the potential failure modes of the structural elements of frame buildings, which may not significantly affect the model's accuracy.
The main advantage of the IFB model is in its computational efficiency and computational robustness, which means that it can be applied to large frame buildings and many ground motions. However, further studies are needed to fully understand the capabilities and limitations of the IFB model when applied to plan-irregular buildings. Therefore, further studies will explore the IFB models' possibilities for fragility analyses of plan-irregular buildings structures and then to loss estimations of building portfolios.

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