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Developing a modified IDA-based methodology for investigation of influencing factors on seismic collapse risk of steel intermediate moment resisting frames

Mohammad M. Maddah[a] and Sassan Eshghi*

Structural Engineering Research Center, International Institute of Earthquake Engineering and Seismology (IIEES), No. 21, Arghavan St., North Dibajee, Farmanieh. Tehran, Iran

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Abstract. Incremental dynamic analysis (IDA) widely uses for the collapse risk assessment procedures of buildings. In this study, an IDA-based collapse risk assessment methodology is proposed, which employs a novel approach for detecting the near-collapse (NC) limit state. The proposed approach uses the modal pushover analysis results to calculate the maximum inter-story drift ratio of the structure. This value, which is used as the upper-bound limit in the IDA process, depends on the structural characteristics and global seismic responses of the structure. In this paper, steel mid-rise intermediate moment resisting frames (IMRFs) have selected as case studies, and their collapse risk parameters are evaluated by the suggested methodology. The composite action of a concrete floor slab and steel beams, and the interaction between the infill walls and the frames could change the collapse mechanism of the structure. In this study, the influences of the metal deck floor and autoclaved aerated concrete (AAC) masonry infill walls with uniform distribution are investigated on the seismic collapse risk of the IMRFs using the proposed methodology. The results demonstrate that the suggested modified IDA method can accurately discover the near-collapse limit state. Also, this method leads to much fewer steps and lower calculation costs rather than the current IDA method. Moreover, the results show that the concrete slab and the AAC infill walls can change the collapse parameters of the structure and should be considered in the analytical modeling and the collapse assessment process of the steel mid-rise intermediate moment resisting frames.

Keywords: IDA method; seismic collapse risk; near-collapse limit state; steel moment-resisting frames; AAC infill walls; composite action

1. Introduction

Past earthquakes' experiences showed that the structural collapse played a significant role in financial losses and casualties after any of the events. This occurrence also increases the mortality rate after an earthquake by disrupting the rescue process. Accordingly, the primary concern of decision-makers and the disaster management organizations is to minimize the casualties and financial losses and the socioeconomic consequences caused by the collapse of buildings.

Incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002) is one of the most accurate and reliable approaches of seismic structural analysis which widely used in the collapse assessment process. This method involves a series of nonlinear dynamic time-history analyses for each ground motion record by scaling it to the several levels of intensity measures (IMs). The results of these time-history analyses for one ground motion create the IDA curve that is a plot of the selected IM against the intended engineering demand parameter (EDP). In this method, the near-collapse limit state for each record reaches where the local slope on the IDA curve reaches 20% of the elastic slope, or the EDP attains a predefined code-based rate (FEMA 2000, Vamvatsikos and Cornell 2002). Utilizing the first criterion requires a large number of the nonlinear time-history analysis from the elastic response up to the collapse of the structure. This approach is very time-consuming and has a high calculation cost (Eshghi *et al*. 2020). On the other hand, the code-based predefined rates of EDPs are proposed for a wide range of buildings and is independent of the investigated structure. So, this criterion is not accurate and overestimates the corresponding spectral collapse capacity (Jalayer *et al*. 2017).

Wu *et al*. (2018) proposed a criterion for detecting the sidesway collapse of special MRFs. In this method, the Arias intensity and the inter-story drift ratio history were used to detect the collapse of the frame (Wu *et al*. 2018). Jalayer *et al*. (2017) suggested that the near-collapse reaches where at least 50%+1 numbers of the columns in each story lose twenty percent of their capacity. Using these criteria is not user-friendly enough for earthquake engineers and requires extensive and time-consuming post-processing. Zahedi and Eshghi (2017) offered a collapse criterion based on the first-mode pushover analysis results. This approach did not consider the higher modes effects and presented under-estimated values for this limit-state. In this paper, a

*Corresponding author, Associate Professor
E-mail: s.eshghi@iiees.ac
[a]Ph.D.
E-mail: mm.maddah@iiees.ac.ir

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novel method is suggested to detect the near-collapse (NC) level in the IDA using the modal pushover analysis results. In the proposed method, the average of the inter-story drift values resulted from the NC limit state of the modal pushover analysis is considered as the upper-bound for the maximum inter-story drift ratio (IDRmax) in the IDA approach. This procedure presents a global measure of the seismic behavior of the structure and is unique for every structure. The number of the required nonlinear dynamic analyses are decreased considerably by using this method. So, the calculation costs and analysis time decrease noticeably rather than using the Vamvatsikos and Cornell (VC) criterion, which is commonly used in the IDA analysis.

Here, the proposed modified IDA approach is used in the collapse risk assessment process. In this procedure, the collapse risk parameters are obtained including the mean annual frequency of collapse (λc) and the probability of one collapse in the remained lifetime of the structure (Eads *et al*. 2012, 2013a).

Mid-rise steel moment-resisting frames (MRFs) constitute a significant part of the conventional buildings in relatively high and high seismic countries such as Iran. In these regions, the seismic design is performed in two types of steel special and intermediate MRFs (AISC 358-16; BHRC 2014). The strong-column/weak-beam (SCWB) philosophy is respected in the seismic design process of special MRFs, but in some seismic design codes, this criterion is not mandatory for the intermediate MRFs (IMRFs). This criterion minimizes occurring the weak column mechanism in the structure and decreases its impact on the overall seismic performance of MRFs (Elkady and Lignos 2014, Eshghi and Maddah 2019). In the current study, three 5, 8, and 11-story IMRFs are selected as case studies, and their collapse risk is investigated by the proposed methodology. The collapse assessment of the structures required comprehensive models that consider all of the effective elements for demonstrating structural behavior. The previous studies showed that the concrete floor slab in the metal deck ceiling system and the presence of infill walls within the frames could considerably affect the collapse of special MRFs.

The composite effects in the metal deck ceiling system increase the beam moment inertia and decrease the SCWB ratios and also changes the inelastic parameters of beam hinges and panel zones (Lignos *et al*. 2011; Elkady and Lignos 2014). These effects in special MRFs has investigated by several kinds of research. More recently, a full-scale collapse test was conducted on a four-story steel special MRFs at the E-Defense facility in Japan (Suita *et al*. 2007). The composite action effects were recognized as one of the primary collapse reasons for this building (Lignos *et al*. 2013). Due to the necessity for modeling the composite action effects in the analytical model, Lignos *et al*. (2011) presented needed relationships for other than RBS beams, and Elkady and Lignos (2014) presented those for RBS beams and the panel zones. These relations have been proposed using the modified Ibarra-Medina-Krawinkler (IMK) model for nonlinear analytical modeling of the structure (Lignos and Krawinkler 2011). These studies indicate that in composite steel beams (Elkady and Lignos 2014): (i) the steel beams flexural strength would increase about 10% to 25%; (ii) the strong-axis moment of inertia is about 40% larger than the bare steel beam; (iii) the cyclic strength and stiffness deterioration becomes asymmetric; (iv) the shear force demand on the panel zone increases. In the current paper, the composite effects are investigated on the seismic collapse risk of the studied IMRFs.

Interaction between masonry infill walls and the frames and its contribution to the seismic performance of special MRFs has been broadly explored in the literature. These researches show that infill walls can reduce displacement demand and decrease the seismic damages, however, the infill-frame interaction may cause local failures in the ends of columns of the joints (Ravichandran and Klingner 2012a; Cavaleri *et al*. 2017; Di Trapani and Malavisi 2019). The stiffer infill walls attract higher seismic loads, which resist as long as the infill walls stayed elastic. After degrading the infill walls, the remaining frames may be collapsed due to lack of resistance and also, brittle and abrupt elimination of the infills (Ravichandran and Klingner 2012a). Recent literature on this topic demonstrated the rising necessity for precise modeling of infilled frames to make valid seismic collapse assessment of structures (Asteris *et al*. 2017; Di Trapani *et al*. 2018). Despite of many different modeling approaches, the most practical method is modeling the masonry infill walls by two equivalent diagonal struts (Di Trapani and Malavisi 2019). The autoclaved aerated concrete (AAC) masonry infill wall could be an appropriate alternative for clay bricks masonry infills. The research showed that this type of masonry construction has appropriate behavior under the cyclic loading, such a way that remained stable without abrupt changes with no out-of-plane instability (Ravichandran and Klingner 2012b). This paper assesses the effects of the AAC infill wall with uniform distribution on the seismic collapse risk of the studied IMRFs.

2. Numerical models

In this study, three 5, 8, and 11-story MRFs with intermediate ductility are designed and utilized as the case studies (Code No.10 2013; Code No.6 2013; BHRC 2014). These buildings are supposed to be located in Tehran, far from active faults, in a very high seismic hazard region and soil type II site (375 m/s < Vs (30 m) < 750 m/s). The effective seismic mass and weight of each story are 4000 kg/m and 48 kN/m, respectively. The elastic modulus of steel is supposed 200 GPa, and the beams and columns yield stresses are considered 235 MPa and 350 MPa, respectively. The concrete slab properties include drib and ts are supposed to be 7 cm and 8 cm, correspondingly. Fig.1 and Table 1 demonstrate the elevation view, section properties, and member details of the structures.

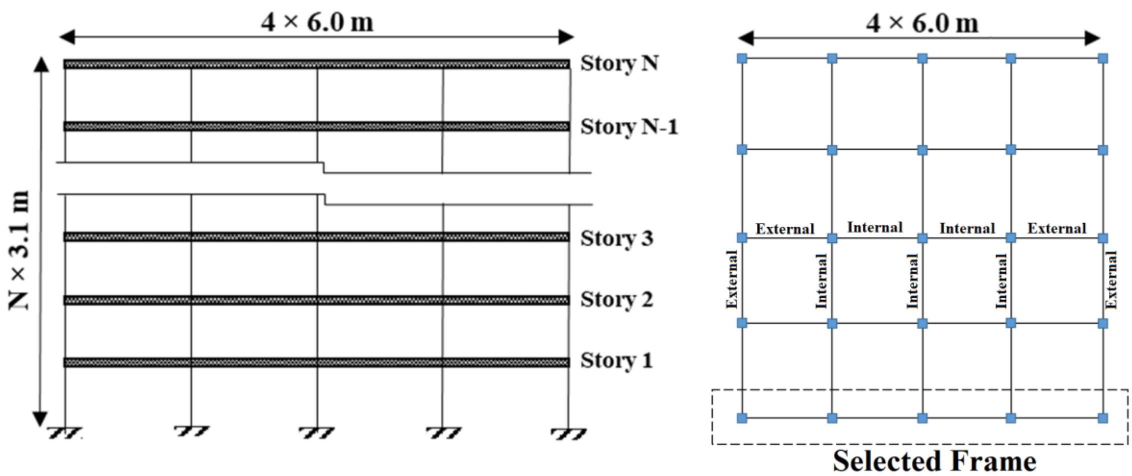
In Table 2 three elastic periods and effective modal mass ratios of the studied structures in three conditions of Bare (without concrete slab and infill wall), Composite (with concrete slab and without infill wall) and Infilled (with concrete slab and AAC infill wall) are presented.

Table 1 Beams and columns configurations of the understudied MRFs

			Story Level										
Model	Element	Position	1	2	3	4	5	6	7	8	9	10	11
5-Story	Beam	External	B2	B2	B3	B3	B4						
		Internal	B3	B3	B3	B3	B4						
	Column	External	C4	C4	C4	C5	C5						
		Internal	C4	C4	C5	C5	C5						
8-Story	Beam	External	B1	B1	B2	B2	B2						
		Internal	B3	B3	B3	B3	B3						
	Column	External	C3	C3	C3	C4	C4						
		Internal	C3	C3	C4	C4	C4						
11-Story	Beam	External	B1	B1	B1	B1	B1	B1	B3	B3	B3	B3	B5
		Internal	B1	B1	B2	B2	B3	B3	B3	B3	B3	B3	B5
	Column	External	C1	C1	C1	C3	C3	C3	C3	C4	C4	C4	C4
		Internal	C2	C2	C2	C3	C3	C3	C3	C4	C4	C4	C4

Table 2 Three first periods and the effective modal mass ratios of the models

Model	Type	T_1 (s)	Mass%	T_2 (s)	Mass%	T_3 (s)	Mass%
5-Story	Bare	1.27	83%	0.44	12%	0.25	3%
	Composite	1.21	83%	0.42	12%	0.24	3%
	Infilled	0.47	84%	0.17	12%	0.09	2%
8-Story	Bare	1.85	81%	0.66	13%	0.39	4%
	Composite	1.76	81%	0.63	13%	0.37	4%
	Infilled	0.74	82%	0.27	13%	0.15	3%
11-Story	Bare	2.16	78%	0.8	14%	0.46	6%
	Composite	2.02	78%	0.75	14%	0.43	6%
	Infilled	1.06	80%	0.38	14%	0.21	5%



Columns*			Beams**				
Tag	D	t	Tag	d	b_f	t_f	t_w
C1	320	20	**B1**	500	220	20	11
C2	300	20	**B2**	440	200	15	10
C3	250	20	**B3**	400	200	15	10
C4	220	20	**B4**	360	180	12	10
C5	200	20	**B5**	320	160	12	10

* HSS shape: D=total width, t=thickness (millimeters)
** I shape: d=total height, b_f=flange width, t_f=flange thickness, t_f=web thickness (millimeters)

Fig. 1 Plan, elevation view and section properties of the understudied MRFs

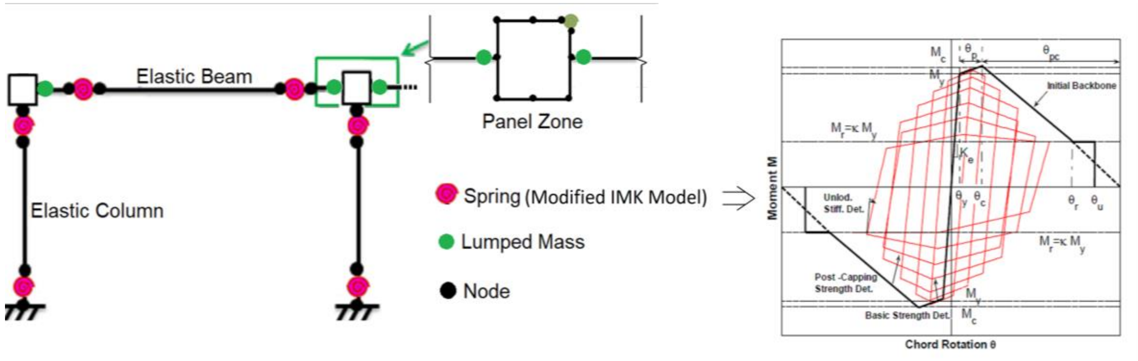


Fig. 2 The OpenSees modeling of MRFs employing a modified IMK deterioration model (Eads *et al*. 2013b)

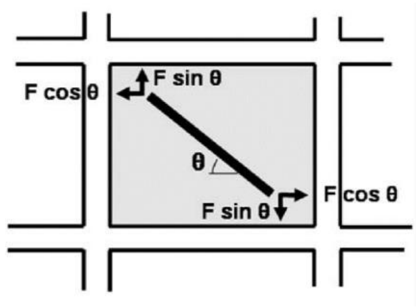
It can be seen that the effects of the higher modes in these frames are unneglectable, and these are magnified in higher buildings.

2.1 Analytical modeling of MRFs

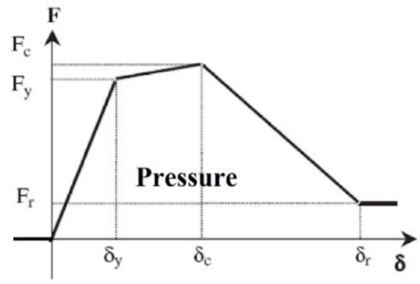
This paper focuses on the sidesway collapse and the vertical collapse mechanism is not considered in the analytical modeling. In the regular mid-rise MRFs with negligible torsion, two-dimensional (2D) analytical models present adequate precision to predict structural collapse (Lignos *et al*. 2013). In this paper, 2D models are produced in the OpenSees program, according to Fig. 2. The beams and columns are modeled by elastic beam-column element and plastic hinge rotational springs at member ends that followed a bilinear hysteretic response and included deterioration based on the modified Ibarra-Medina-Krawinkler (IMK) deterioration model (Lignos and Krawinkler 2010, 2011). The IMK model can simulate cyclic deterioration both in strength and stiffness and has been implemented in the OpenSees analysis platform (Eads and Lignos 2012). In this model, the empirical equations have proposed that predict the deterioration modeling parameters include the pre-capping plastic rotation (θp), post-capping rotation capacity (θpc), and the reference cumulative rotation capacity (Λ). This improvement is made based on the statistical evaluation of calibrated moment-rotation diagrams, obtained from tests and using multivariate regression analysis, quantitative information for modeling of effective yield moment (My), post-yield strength ratio (Mc/My), residual strength ratio (r), and ultimate rotation capacity (θu). Further details about the modified IMK model can be found in Lignos and Krawinkler (2010) and (2011). Also, the panel zones are explicitly modeled using the approach of Gupta and Krawinkler (1999) as a rectangle composed of eight very stiff elastic beam-column elements with one rotational spring to represent shear distortions by a tri-linear backbone curve. In the other three corners of the panel zone, a simple pin connection was used to join the elements (Gupta and Krawinkler 1999; Eads and Lignos 2012). The damping matrix is specified using the Rayleigh damping with a damping ratio of 2% for the first and third modes of vibration.

As previously indicated, the slab increases the flexural strength and stiffness of the composite steel beam such a way that may exceed that of the column. After that, local story collapse mechanisms which implicate column plastic hinging maybe happened. Also, this composite action changes the effective depth of the panel zones in the positive loading direction. In order to consider these effects in the analytical modeling, the methods presented by Lignos *et al*. (2013) and Elkady *et al*. (2014) are utilized. In these methods, which have been verified based on E-Defense full-scale shake table collapse tests, the beam hinges and panel zones nonlinear parameters have been modified by coefficients resulted from a large number of experimental tests. Also, the ratio of the composite beams moment of inertia with respect to bare steel beams has been suggested to be 1.4 on average.

It was mentioned before that in this paper the AAC infill wall effects are investigated on the seismic collapse risk of the steel IMRFs. The studied infill wall has 15 cm width



(a)



(b)

Fig. 3 (a) The equivalent strut model of an infill wall,(b) Backbone curve of a compression strut (Ravichandran 2012)

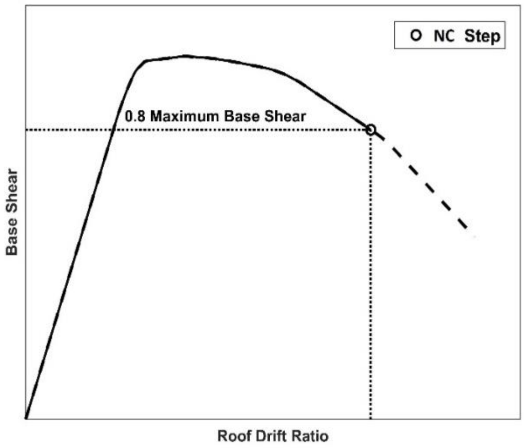


Fig. 4 The NC step in the pushover analysis

and 4.1 MPa compressive strength and has a regular distribution in all frames of the 2D structure.

The method proposed in the Ravichandran and Klinger (2012) research is utilized for analytical modeling of this infill wall (Ravichandran 2012; Ravichandran and Klingner 2012a), which has been presented based on the experimental tests. As shown in Fig. 3, in this modeling method the infill wall section is altered by diagonal compression struts that have connected to the joints. Also, the backbone curve of these compression struts is proposed based on the Ibarra-Krawinkler hysteretic model (Ibarra *et al*. 2005). This component is modeled by 'corotTruss' element and 'Hysteretic' uniaxial material in the OpenSees. Width of the equivalent strut and the other required parameters details can be found in the referenced documents. In this model, the pinching effects are considered in hysteretic rule, so that the pinching factors for strain and stress are supposed to be equal to 0.5 and 0.15, respectively. It should be noted that in-cycle degradation of stiffness and strength of this masonry infill walls is not considered in this modeling approach.

The masonry infill walls may cause local shear failures in structural members of the frame. This type of failure can be addressed as non-simulated collapse modes. For this purpose, the shear demand of the elements is obtained by summing the appropriate component of the axial force in the equivalent struts to shear force in the elements due to frame action alone. Ravichandran (2012b) suggested that for a specified ground motion during IDA if the shear demand of the elements exceeds their shear capacity at more than one spectral intensity, the non-simulated collapse is considered to happen at the second (higher) of these spectral intensities.

3. Modified IDA method

In this section, the IDA method is modified by using a new method for detecting the near-collapse limit state. In this method, the maximum inter-story drift ratio (IDRmax), which has a direct relation to the structural damages of buildings (Abbasnia *et al*. 2014; Bayat 2018), is considered as the EDP. Also, the 5% damping spectral acceleration corresponding to the fundamental vibration period of the structure Sa (T1-5%) is utilized for the IM value (Baltzopoulos *et al*. 2018).

In the proposed method, the upper-bound limit for the IDRmax is calculated from the modal pushover analysis results. For this aim, the average of the inter-story drift values resulted from the NC limit state of the modal pushover analysis is considered as an upper-bound for IDRmax in the IDA. As can be seen in Fig. 4, the NC limit state in the pushover analysis is defined with a 20% drop in the strength of the structure (FEMA 2009).

The modal pushover analyses are performed by using the force distribution according to Eq. 1 which corresponds to the product of the diagonal mass matrix (M) and the i^{th} natural vibration mode ϕ_i (Brozovič and Dolšek 2014).

$$S_i = M\phi_i \tag{1}$$

In the modal pushover analysis, the inter-story drift values can be extracted from the square root of the sum of the squares (SRSS) combination rule as follows

$$\mathrm{IDR}\,|_{j=1}^{N} = \sqrt{\sum_{i=1}^{m} \mathrm{IDR}_{i,j}^{2}}\,|_{j=1}^{N} \tag{2}$$

Where N is story number, j is the story index, m is the number of the considered modes and $\mathrm{IDR}_{i,j}$ is the inter-story drift value in i^{th} mode and j^{th} story in the NC step. In this method, m is determined in a way that the total effective modal mass of the structure be at least 95% of the actual mass.

Finally, the average of the $\mathrm{IDR}\,|_{j=1}^{N}$ values is considered as the upper-bound for the NC limit state (IDR_{NC}) in the IDA approach

$$\mathrm{IDR}_{NC} = \frac{\sum_{j=1}^{N} \mathrm{IDR}_j}{N} \tag{3}$$

The spectral collapse capacity for a specified ground motion record is obtained by taking the intensity measures (IM) value at which the IDR_{max} be higher than IDR_{NC}. If the suggested approach is used, the spectral collapse capacity could be found with significantly fewer steps rather than the VC criterion, which was presented in the introduction. In this method, the determined NC level is unique for the evaluated building and is just dependent on the structural parameters and its seismic behavior.

For using the proposed modified IDA approach, the following steps can be performed (the solution algorithm is inspired by Hunt and Fill approach (Vamvatsikos 2007))

1) The frame is analyzed under the specified record, without any scaling (SF_1=1), and the maximum inter-story drift is determined (IDR_1).

2) The second scale factor is determined by $SF_2=IDR_{NC}/IDR_1$.

3) In all steps (i>2):

3.1) If IDR_i becomes infinity or becomes more than 0.10, the next scale factor will be calculated by using $SF=(SF_i-SF_{i-1})/2+SF_i$.

3.2) If IDR_i becomes less than IDR_{NC}, the next scale factor will be calculated based on IDR_{NC}/IDR_i value where $IDR_i < IDR_{NC}$.

3.3) If IDR_i becomes more than IDR_{NC}, the next scale factor will be calculated by using $SF=SF_i-0.10-0.05\times(i-2)$.

4) The process continues up to where $|IDR_{NC}-IDR_i|/IDR_{NC} < 0.05$.

5) The spectral collapse capacity can be found accurate enough by interpolating between two last spectral acceleration values based on IDR_{NC}.

4. Collapse risk assessment approach

In this study, the collapse risk is evaluated by the mean annual frequency of collapse (λ_c) parameter. Determining λ_c requires two components: the seismic hazard curve and the structure's collapse fragility curve. The hazard curve gives the mean annual frequency of exceeding ground motion intensities at a specific site. The collapse fragility curve describes the structure's probability of collapse conditioned on the intensity of the ground motion. According to Eq. (4) for computing λ_c, the structure's collapse fragility curve is multiplying over the slope of the site's seismic hazard curve. This relation can be solved by a numerical approach, according to Eqs. (5) and (6) (Eads *et al*. 2013a).

$$\lambda_c = \int_0^\infty P(C\,|\,im).|\,d\lambda_{IM}(im)\,| \tag{4}$$

$$\lambda_c = \int_0^\infty P(C\,|\,im).|\,\frac{d\lambda_{IM}(im)}{d(im)}\,|\,d(im) \tag{5}$$

$$\lambda_c = \sum_{i=1}^\infty P(C\,|\,im_i).|\,\frac{d\lambda_{IM}(im_i)}{d(im)}\,|\,\Delta im \tag{6}$$

Collapse fragility curves are calculated by assuming a lognormal distribution for the resulted spectral collapse capacity from structural analysis. In this study, due to the far-fault site of the understudy models, the FEMA P695 far-fault earthquake record catalog is used to perform the IDA. This catalog includes 44 normalized earthquake records which are widely used in the literature. The spectral collapse capacity value is obtained by the proposed near-collapse level criteria. Also, the related standard deviation is equaled to the total uncertainty of the system, which is determined by the FEMA P695 simple approach. According to this guideline, the total uncertainty depends on record-to-record uncertainty (β_{RTR}), design requirements uncertainty (β_{DR}), test data uncertainty (β_{TD}), and modeling uncertainty (β_{MDL}). Utilizing this approach and considering good quality for β_{TD} and β_{MDL} and fair quality for β_{DR}, the total uncertainty equals 0.6.

The λ_c describes the mean yearly rate of collapse. Assuming the occurrence of earthquakes in time follows a Poisson process, this parameter can be translated to a probability of one collapse over the *t* years, R(t) (Eads *et al*. 2013a)

$$R(t) = 1 - e^{-\lambda_c t} \tag{7}$$

R(*t*) denotes "the probability that at least one earthquake occurs during *t* years that is strong enough to cause collapse" (FEMA 2015). The design lifetime of the conventional buildings can be assumed 50 years. If *t* is equal to 50 years, the acceptable value for R(t) is 1% (FEMA 2009, 2015).

Also, considering R(t)=0.01 the remained safety lifetime of the buildings (t_R) can be approximately determined from Eq. (8).

$$t_R = \frac{0.01}{\lambda_c} \tag{8}$$

The steps of the proposed collapse risk assessment methodology are presented in Fig. 5 as a block diagram.

5. Results

The collapse risk assessments of the studied buildings comprise 'Bare, 'Composite' and 'Infilled' frames are carried out by the proposed methodology. According to the effective modal mass ratios of the studied models, presented in Table 2, two modes are considered in the pushover analysis of the 5 and 8-story models and three modes are considered for the 11-story model. The pushover curves and related NC steps are demonstrated in Fig. 6. The upper-bound values for inter-story drift (IDRNC) resulted from the proposed criterion are presented in Table 3.

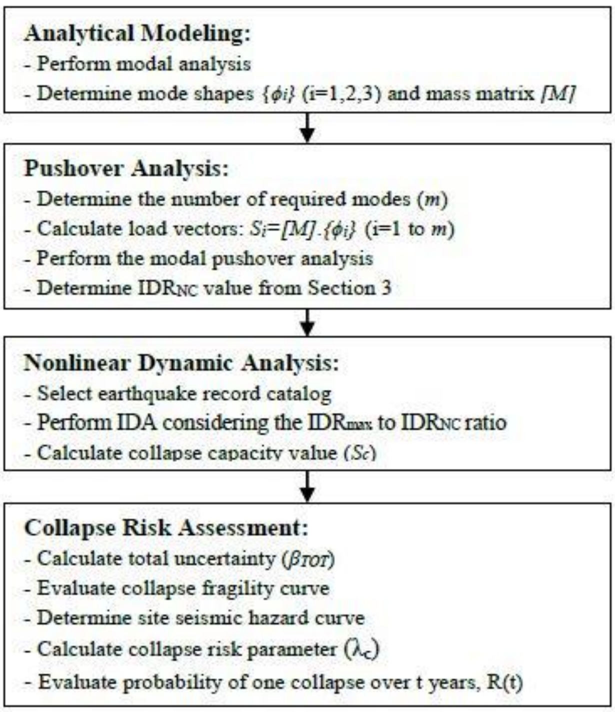
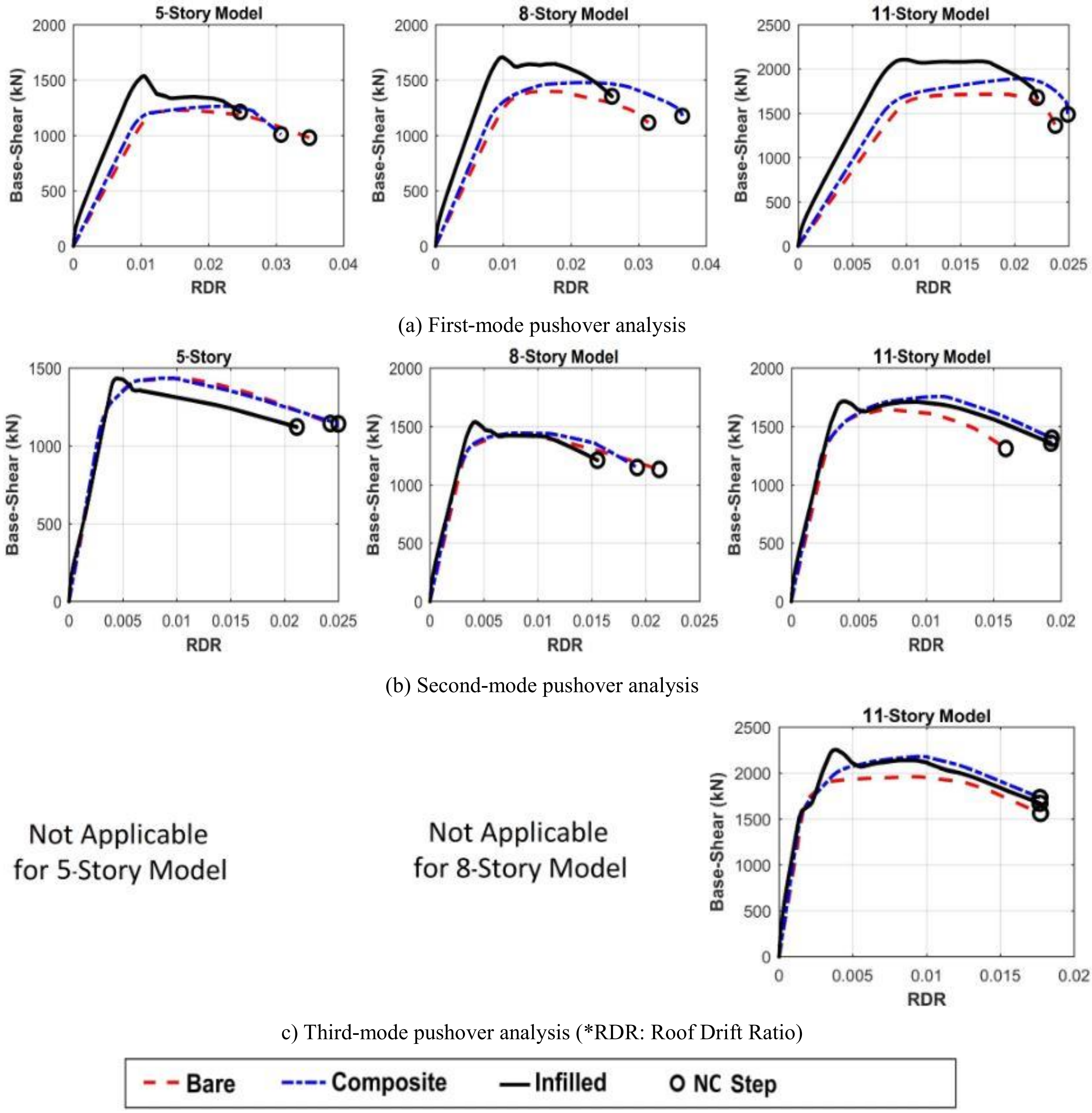


Fig. 5 Block diagram of the modified IDA-based collapse risk assessment method



(a) First-mode pushover analysis

(b) Second-mode pushover analysis

c) Third-mode pushover analysis (*RDR: Roof Drift Ratio)

Fig. 6 Pushover curves and related NC steps of the models: (a) First-mode pushover analysis (b) Second-mode pushover analysis (c) Third-mode pushover analysis (*RDR: Roof Drift Ratio)

The spectral collapse capacity values of the understudy models obtained from the proposed NC criterion and the VC criterion are presented in Table 4.

As the above tables show, in all models the spectral collapse capacity values resulted from the proposed criterion are very close to those that are resulted from the VC criterion. Moreover, these results are more conservative rather than the VC criterion outcomes. These results confirm that the suggested collapse criterion presents acceptable accuracy. Also, this method is easy to implement and considerably reduces the required IDA steps and the calculation costs.

The collapse risk parameters are evaluated by the proposed methodology for the studied models. The collapse fragility curves of the studied frames are determined by the spectral collapse capacity values obtained from the previous section and supposing 0.6 for the total uncertainty. Due to the location of the case study models, the hazard curves of the intended T_1 are determined extrapolation or interpolation of Tehran hazard curves for T_1=0.2 s and T_1=1 s (Mirzaee *et al*. 2012) which is shown in Fig. 7.

Table 3 The IDRNC values for the models, obtained from one mode and SRSS of two or three modes results

	Type	Bare			Composite			Infilled		
Model	Modes	1 Mode	2 Modes	3 Modes	1 Mode	2 Modes	3 Modes	1 Mode	2 Modes	3 Modes
5-Story	IDRNC	0.035	0.062	-	0.031	0.058	-	0.025	0.047	-
8-Story	IDRNC	0.031	0.053	-	0.037	0.056	-	0.027	0.027	-
11-Story	IDRNC	0.025	0.040	0.056	0.026	0.044	0.060	0.023	0.042	0.052

Table 4 Spectral collapse capacity values resulted from the proposed and the VC approaches

Model	Approach	Bare	Composite	Infilled
5-Story	Proposed	0.82 g	0.78 g	0.97 g
	VC	0.84 g	0.80 g	1.01 g
8-Story	Proposed	0.45 g	0.44 g	0.59 g
	VC	0.48 g	0.47 g	0.60 g
11-Story	Proposed	0.43 g	0.46 g	0.65 g
	VC	0.44 g	0.48 g	0.66 g

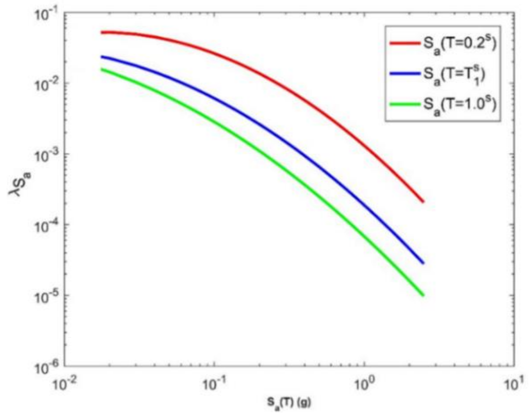


Fig. 7 Tehran hazard curves for T1=0.2 s and T1=1 s (Mirzaee *et al*. 2012).

Table 5 Collapse risk parameters of the models

Model	Type	T_1 (s)	λ_c	$R(t)_{50}$	t_R (year)
5-Story	Bare	1.27	0.000132	0.0066	76
	Composite	1.21	0.000135	0.0067	74
	Infilled	0.47	0.000137	0.0068	73
8-Story	Bare	1.85	0.000161	0.0080	62
	Composite	1.76	0.000167	0.0083	60
	Infilled	0.74	0.000190	0.0094	53
11-Story	Bare	2.16	0.000150	0.0075	67
	Composite	2.02	0.000152	0.0076	66
	Infilled	1.06	0.000164	0.0081	61

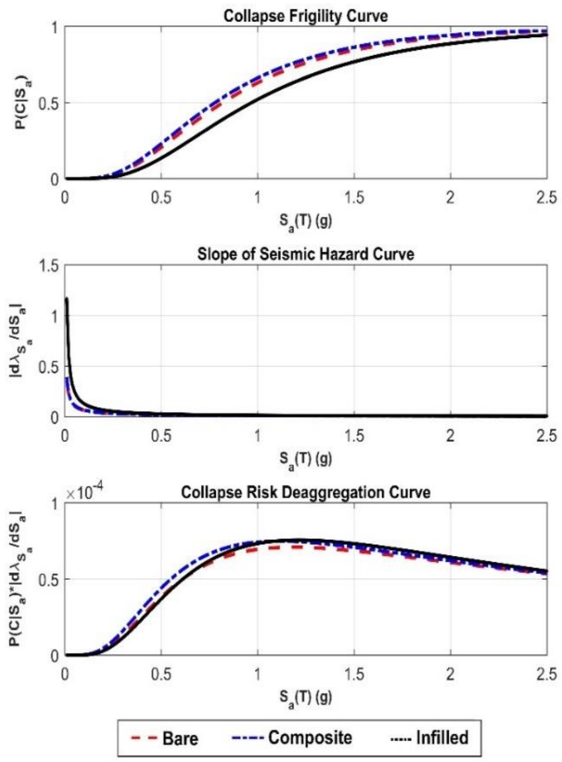


Fig. 8 The collapse risk evaluation of the 5-story models by the deaggregation method

The collapse risk evaluation process of the case study models is shown in Fig. 8 to Fig. 10 and the obtained values of the mean annual frequency of collapse (λ_c) are presented in Table 5. Moreover, the probability of one collapse in 50 years, $R(t)_{50}$, and the remained safety lifetime of the buildings (t_R) is presented in this table.

As mentioned in the introduction, the SCWB rule is not controlled for the understudy IMRFs. These ratios for interior joints of the studied bare frames are presented in Table 6. These values are obtained by AISC Seismic Provisions (ANSI/AISC 2016). As it is shown in Table 6, in the 5-story frame the SCWB ratios have the least values rather than 8 and 11-story frames (the SCWB ratio in all joints are less than 0.9). This issue can be named as the major reason for the different performance of the 5-story frame. So that, the composite action decreases the IDR_{NC} and the spectral collapse capacity in this structure. In the 11-story frame, the SCWB ratios are almost near 1 (between 0.88 and 1.06). Thus, the composite action has positive effects on the structural responses in this model. Furthermore, in the 8-story frame, the SCWB ratios have high dispersion between 0.72 and 1.16. Meanwhile, the IDR_{NC} increases but the spectral collapse capacity almost remains constant, due to the composite action. Moreover, Table 5 shows that the composite action has adverse effects on the collapse risk parameters of the studied models, so that the λ_c and $R(t)_{50}$ increase and the t_R decrease slightly.

Table 6 SCWB ratios of the bare models

	5-Story Bare Model			8-Story Bare Model			11-Story Bare Model		
	Axis 2	Axis 3	Axis 4	Axis 2	Axis 3	Axis 4	Axis 2	Axis 3	Axis 4
Story 1	0.84	0.89	0.84	0.83	1.16	0.83	0.97	0.97	0.97
Story 2	0.77	0.81	0.77	0.74	1.03	0.74	0.97	0.97	0.97
Story 3	0.72	0.72	0.72	0.86	0.91	0.86	1.06	1.06	1.06
Story 4	0.72	0.72	0.72	0.86	0.91	0.86	0.88	0.88	0.88
Story 5	0.55	0.55	0.55	0.77	0.82	0.77	0.95	0.95	0.95
Story 6				0.72	0.72	0.72	0.95	0.95	0.95
Story 7				0.72	0.72	0.72	1.02	1.02	1.02
Story 8				0.55	0.55	0.55	0.91	0.91	0.91
Story 9							0.91	0.91	0.91
Story 10							0.91	0.91	0.91
Story 11							0.65	0.65	0.65

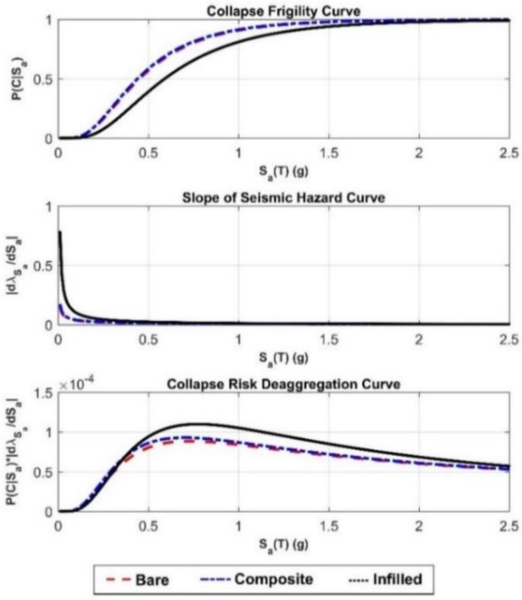


Fig. 9 The collapse risk evaluation of the 8-story models by the deaggregation method

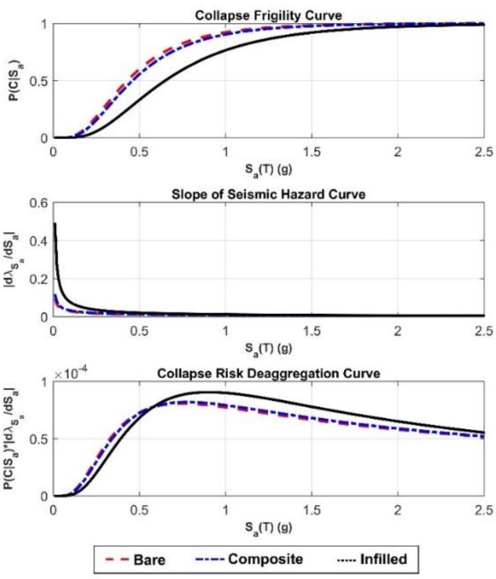


Fig. 10 The collapse risk evaluation of the 11-story models by the deaggregation method

The result confirmed that in the collapse assessment of IMRFs and metal deck ceiling system, it is necessary to consider the composite action of the concrete slab and steel beams in the analytical model.

For investigating the behavior of the AAC infilled frames, infill strength ratio (R_τ) is defined by Ravichandran (2012) as the ratio of the lateral in-plane strength infill walls to the story shear strength of the bare frame. The story shear strength is obtained using a story mechanism that leads to hinges at the top and bottom of columns at that story (Fig.11)

$$R_\tau = \frac{F_{infill}}{F_{story}} = \frac{\sum 152\ t_{infill} f_m}{\frac{1}{h_{st}} \sum Z_c F_{yc}} \tag{9}$$

Where t_{infill} is the infill wall thickness, f_m is the compressive strength of the infill wall material, Z_C is the column plastic modulus, F_{yC} is the column yield stress, and h_{st} is the height of the story. The remained parameters are introduced previously. According to Ravichandran and Klingner (2012) investigations, R_τ values higher than 0.35 are associated with progressive deterioration of seismic performance and leading to story mechanisms concentrated in the lower stories and local shear failures in the frame members. While R_τ ratio less than 0.35 could have positive effects on the collapse probability and does not change the failure mechanism due to frame-infill interaction forces (Ravichandran and Klingner 2012a). In the current study, this ratio is determined for understudy infilled frames, which are presented in Table 7.

Obviously, the R_τ values are less than 0.35 in all stories of the studied frames. Also, it is predicted that the presence of the AAC infill wall will not change the failure mechanism of the frames. The obtained results confirm this issue so that in the studied frames the non-simulated collapse due to local shear failure has not been occurred.

On the other hand, it is observed in Table 4 that the presence of AAC infill increases the spectral collapse capacity values for about 25% to 40% comparing to the bare frame cases, although impairs the collapse risk parameters. In these frames, the collapse risk (λ_c) and the

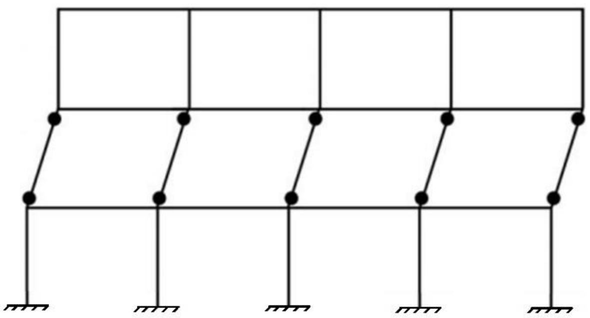


Fig. 11 The story mechanism used to calculate story shear strength of the bare frame (Ravichandran 2012)

Table 7 Rτ ratios of the models

	No. of Stories										
Model	1	2	3	4	5	6	7	8	9	10	11
5-story	0.23	0.23	0.26	0.28	0.28						
8-story	0.17	0.17	0.20	0.23	0.23	0.23	0.28	0.28			
11-story	0.11	0.11	0.11	0.17	0.17	0.17	0.17	0.17	0.23	0.23	0.23

collapse probability $R(t)_{50}$ are increased, and the remained lifetime safety (t_R) is decreased.

As a final point, it is observed that the collapse risk and collapse probability values of the studied frames are less than 0.001 and 0.01, respectively. It shows that the studied models have acceptable seismic performance and are not categorized as high-risk buildings

6. Conclusions

Here, an attempt is made to develop a new IDA-based seismic collapse assessment methodology for the intermediate steel moment-resisting frames (IMRFs). For this purpose, a modified IDA approach is used in which the near-collapse state occurs when the maximum inter-story drift exceeds an upper-bound limit. This limit depends on the structural characteristics, and the seismic responses based on the modal pushover analysis.

The composite action of the concrete slab and steel beams and panel zones could change the failure mechanism of the MRFs, especially in frames in which the strong-column/weak-beam rule is not considered. So the collapse risk is studied by the inclusion of these effects in the ceiling system. Furthermore, the effects of the AAC infill walls on the collapse risk of these frames are investigated. Both of these attributes are included in the newly developed approach.

Three 5, 8, and 11-story steel MRFs located far from active faults are designed and selected for implementing the proposed method. These frames are assessed in three situations of (i) bare, (ii) with concrete slab, and (iii) with concrete slab and AAC infill walls. The comparison of the results from the proposed method and the VC method, which its application is prevalent, shows that the proposed approach is accurate enough to evaluate the spectral collapse capacity of the structures. Utilizing this method leads to fewer steps in the IDA analysis and can decrease the analysis time and calculation costs. Results of the performed analysis show that the composite action of the concrete slab and the steel beams and panel zones can increase the collapse risk of steel IMRFs and must be considered in the analytical modeling stage. Also, the results demonstrated that the AAC infill wall increases the collapse risk of the studied IMRFs.

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