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Evaluation of seismic collapse capacity of regular RC frames using nonlinear static procedure

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Abstract. The Incremental Dynamic Analysis (IDA) procedure is currently known as a robust tool for estimation of seismic collapse capacity. However, the procedure is time-consuming and requires significant computational efforts. Recently some simplified methods have been developed for rapid estimation of seismic collapse capacity using pushover analysis. However, a comparative review and assessment of these methods is necessary to point out their relative advantages and shortcomings, and to pave the way for their practical use. In this paper, four simplified pushover analysis-based methods are selected and applied on four regular RC intermediate moment-resisting frames with 3, 6, 9 and 12 stories. The accuracy and performance of the different simplified methods in estimating the median seismic collapse capacity are evaluated through comparisons with the results obtained from IDAs. The results show that reliable estimations of the summarized 50% fractile IDA curve are produced using SPO2IDA and MPA-based IDA methods; however, the accuracy of the results for 16% and 84% fractiles is relatively low. The method proposed by Shafei *et al.* appears to be the most simple and straightforward method which gives rise to good estimates of the median sidesway collapse capacity with minimum computational efforts.

Keywords: seismic collapse capacity; IDA method; nonlinear static analysis procedure; RC moment-resisting frames

1. Introduction

The main objectives of the modern building codes are to safeguard the life safety of building occupants and to prevent the structural collapse during strong earthquakes. A sidesway collapse may trigger due to large interstory drifts that are amplified by P-delta (second order) effects, in conjunction with monotonic and cyclic strength and stiffness deterioration of structural components. To effectively prevent the sidesway structural collapse, the collapse process and the failure modes of structures must be properly predicted. This helps the engineer to predict the target reliability index and the actual rate of structural damages (Ghasemi and Nowak, 2017). Several shaking-table tests (e.g., Kabeyasawa and Sanada 2001, Elwood 2002, Kanvinde 2003, Rodgers and Mahin 2006, Yavari and Elwood 2009, Lignos *et al*. 2011) have been used to understand the fundamental sidesway collapse mode of buildings. However, these tests are often very expensive and difficult to carry out for the full-scale buildings. As an alternative, numerical simulation is widely used to study the collapse response. Several researchers have used a step-by-step finite element analysis to assess the sidesway collapse response of building structures (Challa and Hall 1994, Martin and Villaverde 1996, Mehanny and Deierlein 2001, Lu *et al*. 2013). The results obtained from these analyses show that the methods are relatively reliable but extremely time-consuming and unsuitable for practical applications (Villaverde 2007). A number of research studies have also focused on the assessment of seismic collapse response of buildings by using single-degree-of-freedom (SDOF) models (e.g., MacRae 1994, Bernal 1998, Ibarra *et al*. 2002, Williamson 2003, Miranda and Akkar 2003, Ibarra and Krawinkler 2011). Although these models are simple but their results are not sufficiently reliable; because as shown in the previous studies (e.g., Bernal 1998), the seismic collapse response of a structure strongly depends on the shape of its failure mechanism; whereas this shape cannot be taken into account by such simple models. On the other hand, several researches (e.g., Medina and Krawinkler 2003, Ibarra and Krawinkler 2005, Zareian and Krawinkler 2009) have aimed at predicting the seismic collapse response of multi-degree-of-freedom (MDOF) structures by using Incremental Dynamic Analysis (IDA) procedure (Vamvatsikos and Cornell 2002). The results show that there is a good correlation between the IDA and experimental test results. Nonetheless the IDA method requires a large number of nonlinear response history analyses and therefore is time-consuming and less popular among practicing engineers. Recognizing the complexity and difficulties involved in the IDA method, several nonlinear static (pushover) analysis-based methods have recently been developed to estimate the seismic collapse response of building structures with less computational effort. A brief review of these methods follows.

Static pushover to incremental dynamic analysis (SPO2IDA) procedure was developed (Vamvatsikos and

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Cornell 2005) to convert the capacity (pushover) curve of the first-mode-dominant building structures into 16^{th}-percentile, 50^{th}-percentile, and 84^{th}-percentile IDA curves. The procedure first idealizes the pushover curve of the structure with a quadri-linear backbone curve including a post-yield hardening region, a negative stiffness segment plus a final residual plateau that terminated with a drop to zero strength. The method then uses nonlinear time history analysis results of the equivalent SDOF system with the constructed backbone curve to approximate the 16%, 50% and 84% fractile IDA curves. In another research, an approximate method based on modal pushover analysis (MPA-based IDA) procedure was developed (Han and Chopra 2006) to estimate the collapse potential of buildings with less computational effort. The procedure estimates the seismic demands due to each ground motion at each intensity level by using nonlinear response history analysis (RHA) of the equivalent SDOF systems instead of using nonlinear RHA of MDOF systems. In another study MPA-based IDA method was used to determine the collapse fragility curve of steel frame structures (Han *et al*. 2010). In this study to avoid the computationally demanding nonlinear RHA, some empirical equations were used by the method to estimate the collapse strength ratio coefficient for the 'first-mode' SDOF system. The results show that MPA-based IDA method can well estimate the seismic collapse capacity and fragility curve of steel frames not only for the first-mode-dominated structures, but also for structures having significant higher mode effects. Some years later, an approximate version of MPA-based IDA method was developed (Moon *et al*. 2012) to further decrease the computational time required by the original MPA-based IDA method by using the empirical equation of the inelastic displacement ratio instead of using nonlinear RHA of the equivalent SDOF systems. A good correlation was observed between the results obtained from approximate MPA-based IDA method with those given by the original MPA-based IDA and exact IDA approaches. Another approximate pushover analysis-based procedure has also been proposed by Liel and Tuwair (2010) to estimate the median seismic collapse capacity of building structures. The proposed method benefits from nonlinear static analysis to obtain an initial estimate of seismic collapse capacity of the structure. Then, the final median seismic collapse resistance is obtained by following an iterative algorithm. In another study, a simplified approximate pushover analysis-based method was developed to estimate the median value and the dispersion of seismic sidesway collapse capacity of moment-resisting frame and shear wall structural systems by using generic MDOF models (Shafei *et al*. 2011). The proposed method is different from the previous studies in which an equivalent SDOF system was utilized to estimate the seismic collapse capacity. Nonetheless, due to the lack of experimental data for collapse response of shear wall structures, the accuracy of the proposed method has not been demonstrated for these buildings. The "collapse capacity spectrum" method has been proposed by Adam and Jäger (2012) for the seismic collapse assessment of inelastic non-deteriorating moment-resisting frames vulnerable to P-delta effects. The proposed method requires two nonlinear static analyses with and without gravity loads to estimate global hardening and post-yielding stiffness ratio. In another investigation, a web-based methodology for the prediction of summarized IDA curves of the first-mode-dominant structures was developed (Peruš *et al*. 2012), which requires seven parameters, where five of them describe the idealized pushover curve. Recently a simplified analysis procedure was proposed to estimate the seismic collapse margin ratio of frame structures by only using the pushover analysis results (Hamidia *et al*. 2013). The proposed method is based on replacing a MDOF structural model with a fictitious nonlinear SDOF system, characterized by an elastic-perfectly-plastic relationship between lateral force and roof displacement obtained from a standard pushover analysis. Some other pushover-based methods are also available in the literature that can be used for the collapse response assessment of structures. Some of these methods can be found in FEMA-440 (2005) and Yang and Tasnimi (2016).

As the number of simplified nonlinear analysis methods for seismic collapse assessment is increasing in recent years, it is important to identify the potential limitations of these methods and to compare their effectiveness in simulating the seismic collapse response of structures. This will pave the way for possible future standardization of the simplified collapse estimation rules.

The objective of this paper is to evaluate the performance of some simplified methods in predicting the seismic collapse capacity for reinforced concrete frame structures. For this purpose, four different simplified pushover analysis-based methods are selected namely: SPO2IDA method (Vamvatsikos and Cornell 2005), MPA-based IDA method (Han and Chopra 2006) and those proposed by Shafei *et al*. (2011) and Hamidia *et al*. (2013). The first two represent methods that use the concept of equivalent SDOF system with probably less computational efforts as compared with similar methods. On the other hand, the methods proposed by Shafei *et al*. and Hamidia *et al*. both use closed-form equations, representing the simplest methods available for the rapid estimation of seismic collapse response of structures. The capability of these methods to estimate the median and the important IDA fractiles of seismic collapse capacity of RC structures is investigated through comparisons with benchmark results obtained from a comprehensive set of IDAs. Four intermediate reinforced concrete moment-resisting frames that incorporate deterioration of components are considered and a set of twenty far-field ground motion records are used for this assessment.

2. Selected simplified pushover analysis-based methods

In this section an overview of the selected simplified pushover analysis-based methods is presented.

2.1 SPO2IDA method

The Static Pushover 2 Incremental Dynamic Analysis (SPO2IDA), which has been developed by Vamvatsikos and Cornell (2005) is considered to be a simplified version of IDA, reduces the computational difficulties in IDA by establishing a connection between the static pushover and IDA methods. The procedure defines the force-displacement relationship of the equivalent nonlinear SDOF system so as to match the pushover curve of the original MDOF structure. The peak displacement of the resulting SDOF system and the summarized IDA curves are then determined using empirical equations implemented in the SPO2IDA software. Details of the implementation are described in Vamvatsikos and Cornell (2005).

2.2 MPA-based IDA method

The modal pushover analysis-based IDA (MPA-based IDA) method, which has been proposed by Han and Chopra (2006) is an approximate method used for evaluating the seismic collapse potential of structural systems. The procedure avoids the computationally demanding IDA in collapse assessment and instead proposes a method which uses modal pushover analysis (MPA) of the structure (Chopra and Goel 2002). As shown in the previous studies (Vamvatsikos and Cornell 2005, Han and Chopra 2006), because the higher modes of vibration have insignificant role in the seismic sidesway collapse resistance of structures, only the first mode effect is typically considered by MPA-based IDA in the seismic collapse response analysis of buildings. In the procedure, an empirical equation for the calculation of collapse strength ratio (R_c) is defined. The equation has been developed for SDOF systems with strength-limited bilinear backbone curves (Ibarra and Krawinkler 2005), which is more appropriate for steel structures. The application of this equation for RC buildings leads to inaccurate collapse capacity estimates. Hence in the current study, the results obtained from the IDA of the equivalent SDOF systems will be used by the MPA-based IDA for the seismic collapse analysis of the buildings. A step-by-step summary of the MPA-based IDA procedure considering only the first mode effect is presented in the following.

(1) Calculate the first-mode (fundamental) natural period, T_1, and the corresponding mode shape vector, φ_1, for the building.

(2) Develop the base shear–roof displacement (V_{b1}-u_{r1}) pushover curve by nonlinear static analysis of the building using the lateral force distribution $\mathbf{s}_1^*=\mathbf{m}\boldsymbol{\varphi}_1$ where $\mathbf{m}$ is the mass matrix.

(3) Idealize the pushover curve as a trilinear backbone curve.

(4) Convert the idealized pushover curve to obtain the force–displacement (F_{s1}/L_1-D_1) relation for the first-'mode' inelastic SDOF system by utilizing $F_{s1}/L_1=V_{b1}/M_1^*$ and $D_1=u_{r1}/\Gamma_1\varphi_{r1}$ in which M_1^* is the first-mode effective mass; φ_{r1} is the value of φ_1 at the roof, and $\Gamma_1=\varphi_1^T\mathbf{m1}/\varphi_1^T\mathbf{m}\varphi_1$.

(5) Estimate the seismic collapse response of the equivalent SDOF system constructed in the previous step by IDA for a set of ground motion records.

(6) Develop the summarized IDA curves of the original MDOF structure by using the results obtained from the previous step and the transformation factors presented in step 4.

2.3 Method proposed by Shafei et al.

Unlike the SPO2IDA and MPA-based IDA methods where the seismic collapse response is obtained by nonlinear RHA of a SDOF system, the simplified method proposed by Shafei *et al*. (2011) uses simple MDOF mathematical models denoted as 'generic structures'. The generic structures considered for modeling of moment-resisting frames consist of elastic beam-column elements with inelastic rotational springs at their both ends. Ranges of variation of different structural parameters selected for the models are based on experimental data that are referenced in other studies (Zareian and Krawinkler 2009, Haselton *et al*. 2007). A comprehensive database of pushover and collapse fragility curves is then developed by nonlinear static analysis and IDA of the models. Accordingly, some closed-form expressions for the rapid estimation of median and dispersion of seismic collapse capacity of structures are developed by multivariate regression analysis of the results. Fundamental period, T, yield base shear coefficient, γ (defined as the ratio of yield base shear to total weight of the building), and the parameters derived from idealized pushover curve are introduced by the proposed method as the most important parameters affecting the dynamic collapse capacity of moment-resisting frame and shear wall structures. The median seismic collapse capacity of a frame structure ($\hat{\eta}_c$) is estimated as follows

$$\ln\left(\hat{\eta}_c\right)=z_0+z_1\left(\alpha_T\right)+z_2\left(\gamma\right)+z_3\ln\left(\Theta_{pl}\right)+z_4\ln\left(\frac{\Theta_{pc}}{\Theta_{pl}}\right) \tag{1}$$

where coefficients z_i, i=0-4 are determined by using multivariate regression analysis and summarized in Table 1 for generic moment-resisting frames. α_T is the ratio of fundamental period of the structure to the number of stories. Θ_{pl} and Θ_{pc} are also the required parameters obtained from the idealized trilinear pushover curve (Fig. 1). Note that the idealization of the pushover curve is carried out based on ASCE-41 (2013) Standard.

2.4 Method proposed by Hamidia et al.

A simplified nonlinear analysis procedure has recently been developed by Hamidia *et al*. (2013) to estimate the seismic sidesway collapse margin ratio (CMR) of frame structures without using the IDA. The procedure is consistent with the FEMA-P695 (2009) methodology and benefits from a comprehensive database of peak displacement response of nonlinear SDOF oscillators for various seismic intensities. The details of the proposed method in estimating the CMR and median seismic collapse capacity of frame structures are as follows:

1. Calculate the elastic fundamental period of the

Table 1 Coefficients of median collapse capacity prediction relationship (Eq. (1)) for generic moment-resisting frames (after Shafei *et al*. 2011)

Coefficient	4-story	8-story	12-story	All
z_0	1.80	2.27	2.49	2.73
z_1	-2.30	-4.54	-5.61	-3.65
z_2	1.76	2.75	3.56	2.26
z_3	0.35	0.48	0.56	0.61
z_4	0.27	0.16	0.13	0.27
R^2	0.87	0.94	0.95	0.93

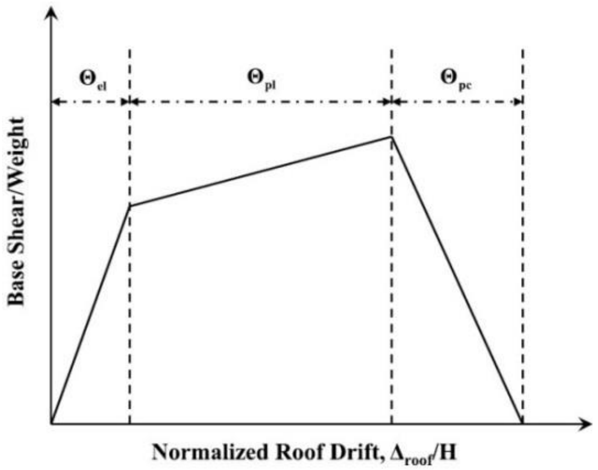


Fig. 1 Definition of parameters required from idealized pushover curve for generic structural models in the method proposed by Shafei *et al*. (2011)

structure, T_{el}, and the corresponding mode-shape, ϕ_1.

2. Perform the nonlinear static analysis of the building using the lateral force distribution $\mathbf{s}_1^*=\mathbf{m}\boldsymbol{\varphi}_1$ ($\mathbf{m}$ is the mass matrix) until a loss of 20% of the base shear capacity is observed.

Idealize the pushover curve as a bilinear curve according to the FEMA-P695 methodology and determine the ultimate roof displacement (δ_u) and target ductility ratio (μ_T) (see Fig. 2).

3. From the pushover analysis, construct the inelastic mode-shape (ϕ_I) using the displacement of each story when the ultimate roof displacement (δ_u) is reached.

4. Calculate the inelastic mode participation factor (Γ_I) as follows

$$\Gamma_I = \frac{\phi_I^T \mathbf{m1}}{\phi_I^T \mathbf{m}\phi_I} \tag{2}$$

where **1** is a vector with all unity components.

5. Extract the reduction factor (r), defined as the ratio of the spectral acceleration at which half of the records cause the bilinear inelastic SDOF oscillator to exceed the target ductility, to the yield pseudo-acceleration, from Table I in Hamidia *et al*. (2013).

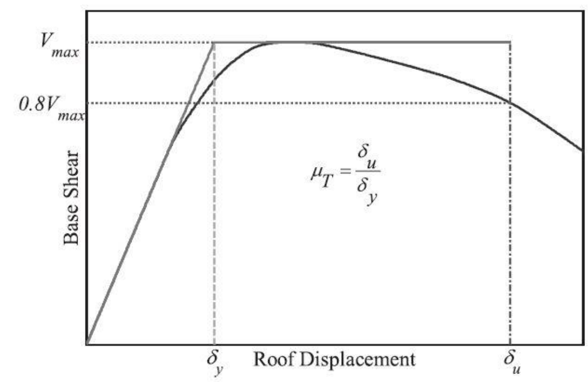


Fig. 2 Definition of parameters required from bilinear idealized pushover curve in the method proposed by Hamidia *et al*. (2013)

6. Calculate the CMR as follows

$$\begin{aligned} T_{el} < T_s &\rightarrow \mathrm{CMR} = \frac{4\pi^2\delta_u \mathrm{r}}{S_{MS}\mu_T T_{el}^2 \Gamma_I \phi_{I,r}} \\ T_{el} \geq T_s &\rightarrow \mathrm{CMR} = \frac{4\pi^2\delta_u \mathrm{r}}{S_{M1}\mu_T T_{el} \Gamma_I \phi_{I,r}} \end{aligned} \tag{3}$$

in which S_{MS} and S_{M1} are the 5%-damped spectral accelerations for maximum considered earthquake (MCE) at short and one second periods, respectively, $\phi_{I,r}$ is the inelastic mode shape component at the roof, and T_s is the characteristic transition period of the earthquake response spectrum.

7. Compute the median seismic collapse capacity of the frame structure ($\hat{S}_{a,\,col}$) as follows

$$\hat{S}_{a,\,col} = \mathrm{CMR} \times S_{MT} \tag{4}$$

where S_{MT} is the 5%-damped MCE spectral acceleration at the fundamental period of the structure.

3. Analytical models and ground motion ensemble

In this section, four RC intermediate moment-resisting frame buildings with 3, 6, 9 and 12 stories are designed and used for the evaluation of the selected simplified nonlinear analysis methods in estimating the median seismic collapse capacity of RC structures. The buildings are located in Los Angeles area with high seismic hazard, and designed in accordance with the ACI 318-11 (2011) and ASCE-7 (2010) requirements. All buildings have similar plan dimensions of 15 m×15 m with 3 bays in each primary direction. Height of the first story is 3.5 m and other stories have a height of 3 m. The dead and live loads are equal to 5.2 and 2 KN/m^2 on the floor area. The seismic mass is assumed to be equal at all floors and consist of the dead load plus 20% of the live load. It is assumed that the lateral

Table 2 Natural periods of RC frame models in this study

Mode	Modal natural periods T_n(s)			
	3-story	6-story	9-story	12-story
1	0.52	0.77	1.07	1.43
2	0.17	0.27	0.39	0.50
3	0.08	0.15	0.23	0.30

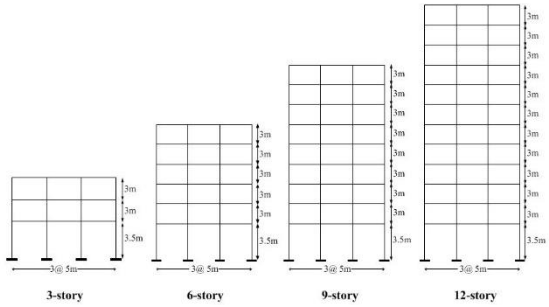
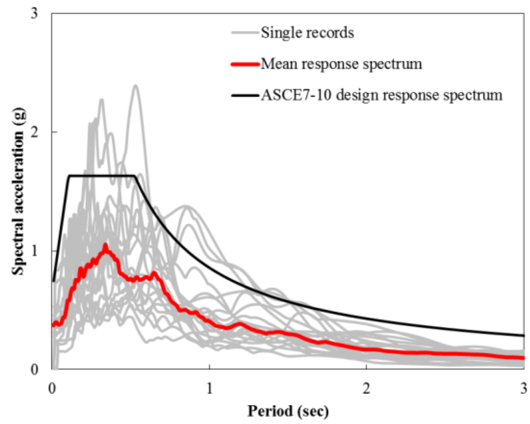


Fig. 3 Elevation view of the structural models considered for this study

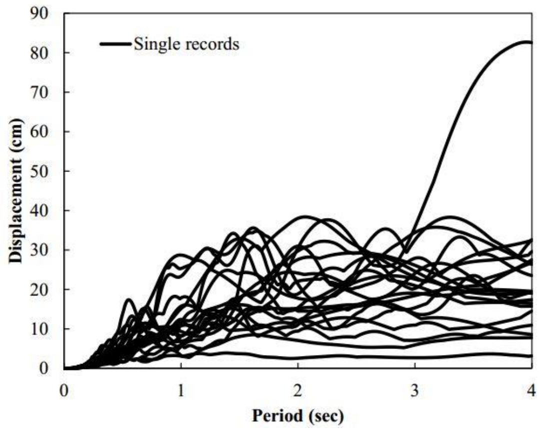
load is resisted by four intermediate RC moment frames in each primary direction. In each case a typical interior frame is considered for the seismic collapse response assessment.

Two-dimensional analytical models are constructed using the OpenSees structural analysis platform (2007) for each building. The beam-column members are modeled by one-component lumped plasticity elements composed of an elastic segment with two concentrated plastic hinges at both ends. The plastic hinges are modeled by nonlinear zero-length rotational springs with stiffness degradation and strength deterioration characteristics as proposed by Ibarra *et al*. (2005). In the present research, the properties of the plastic hinges are predicted from a series of empirical relationships relating RC column design characteristics to modeling parameters and experimental data (Haselton and Deierlein 2007). It is noted that the one-component lumped plasticity element can capture the strain softening associated with rebar buckling and spalling phenomena, which are critical for simulating seismic collapse in RC structures (FEMA-P695 2009). Centerline dimensions are used in the element modeling, and the columns are assumed to be fixed at the base. The effective initial stiffness of beam-column elements is defined using the secant stiffness through 40% of the yield moment. This initial stiffness value can be more appropriate for modeling the full range of seismic performance of structures from small deformations up to collapse (Goulet *et al*. 2007). 5% Rayleigh damping is used for the first and third modes of vibration. P-delta effect is also considered in this study. The elevation of RC frame models along with their first three periods of vibration is presented in Fig. 3 and Table 2, respectively.

A set of twenty records of the FEMA-P695 far-field ground motions is used for the seismic collapse response assessment of the example structures. These ground motions were all recorded on the stiff soil and can be classified as large-magnitude-small-distance (LMSD) ground motion records. Moment magnitudes of these



(a) Pseudo-acceleration spectra



(b) Displacement spectra

Fig. 4 Pseudo-acceleration spectra and displacement spectra of the selected ground motion records, damping ratio=5%. The ASCE-7 design response spectrum for the Los Angeles area is also shown

ground motions vary from 6.5 to 7.5, and closest distances to the fault rupture area are in the range of 12-23.6 km. Thus, no near-fault ground motions with directivity effects are included. The 5%-damped elastic pseudo-acceleration and displacement spectra for each of ground motion records are presented in Fig. 4. The mean pseudo-acceleration spectrum for the selected records together with the ASCE-7 (2010) design response spectrum are also shown in the same figure. More characteristics of the ground motion records are provided in Table 3 (FEMA- P695 2009).

4. Evaluation of collapse response of the frames

In this section, the seismic collapse response of the RC frame structures based on IDA and the selected simplified nonlinear analysis methods are presented and discussed.

4.1 IDA and pushover analysis results

To determine the seismic collapse capacity of the example structures, each of ground motions listed in Table 3 are individually applied to the RC frame models by using IDA approach. The IDA requires a series of nonlinear time-

Table 3 Earthquake ground motions used in this study

NO.	Earthquake	Year	Magnitude	Station name	Component	PGV (cm/s)	PGA (g)
1	Northridge	1994	6.7	Beverly Hills - 14145 Mulhol	MUL009	58.9	0.42
2	Northridge	1994	6.7	Beverly Hills - 14145 Mulhol	MUL279	62.8	0.52
3	Northridge	1994	6.7	Canyon Country - W Lost Cany	LOS000	43.0	0.41
4	Northridge	1994	6.7	Canyon Country - W Lost Cany	LOS270	45.0	0.48
5	Duzce, Turkey	1999	7.1	Bolu	BOL000	56.4	0.73
6	Duzce, Turkey	1999	7.1	Bolu	BOL090	62.1	0.82
7	Imperial Valley	1979	6.5	Delta	H-DLT262	24.9	0.24
8	Imperial Valley	1979	6.5	Delta	H-DLT352	33.0	0.35
9	Imperial Valley	1979	6.5	El Centro Array #11	H-E11140	34.4	0.36
10	Imperial Valley	1979	6.5	El Centro Array #11	H-E11230	42.1	0.38
11	Kobe, Japan	1995	6.9	Shin-Osaka	SHI000	37.8	0.24
12	Kobe, Japan	1995	6.9	Shin-Osaka	SHI090	27.9	0.21
13	Kocaeli, Turkey	1999	7.5	Duzce	DZC180	58.9	0.31
14	Kocaeli, Turkey	1999	7.5	Duzce	DZC270	46.4	0.36
15	Landers	1992	7.3	Yermo Fire Station	YER270	51.4	0.24
16	Landers	1992	7.3	Yermo Fire Station	YER360	29.7	0.15
17	Landers	1992	7.3	Coolwater	CLW-LN	12.6	0.17
18	Landers	1992	7.3	Coolwater	CLW-TR	20.4	0.18
19	Loma Prieta	1989	6.9	Capitola	CAP000	35.0	0.53
20	Loma Prieta	1989	6.9	Capitola	CAP090	29.2	0.44

history analyses and each record is scaled to several levels of intensity to encompass the full range of structural behavior from elastic to collapse (Vamvatsikos and Cornell 2002). The results of these analyses for one ground motion lead to one IDA curve. The IDA curve is a plot of ground motion intensity measure (IM) against an engineering demand parameter (EDP). In this study, the spectral acceleration corresponding to the first mode elastic vibration period of the structure, $S_a(T_1)$, and the maximum interstory drift ratio (MIDR) are chosen as the IM and EDP parameters for the development of IDA curves, respectively. Nonetheless, IDA results based on roof displacement demand are also presented to estimate the median ultimate roof displacement (collapse point) for each frame model.

The IDA results developed based on roof displacement demand will be used for the evaluation of SPO2IDA and MPA-based IDA methods, which both use the equivalent SDOF models. In IDAs, the "sidesway collapse capacity" is defined as the spectral acceleration value at which the structure becomes dynamically unstable due to unbound increase of MIDR or roof displacement. This occurs when the IDA curve becomes flat. The IDA curves for the example structures subjected to the set of twenty ground motion records are shown in Figs. 5 and 6. The median seismic collapse capacities ($\hat{S}_{a,\,col}$) obtained from IDAs are

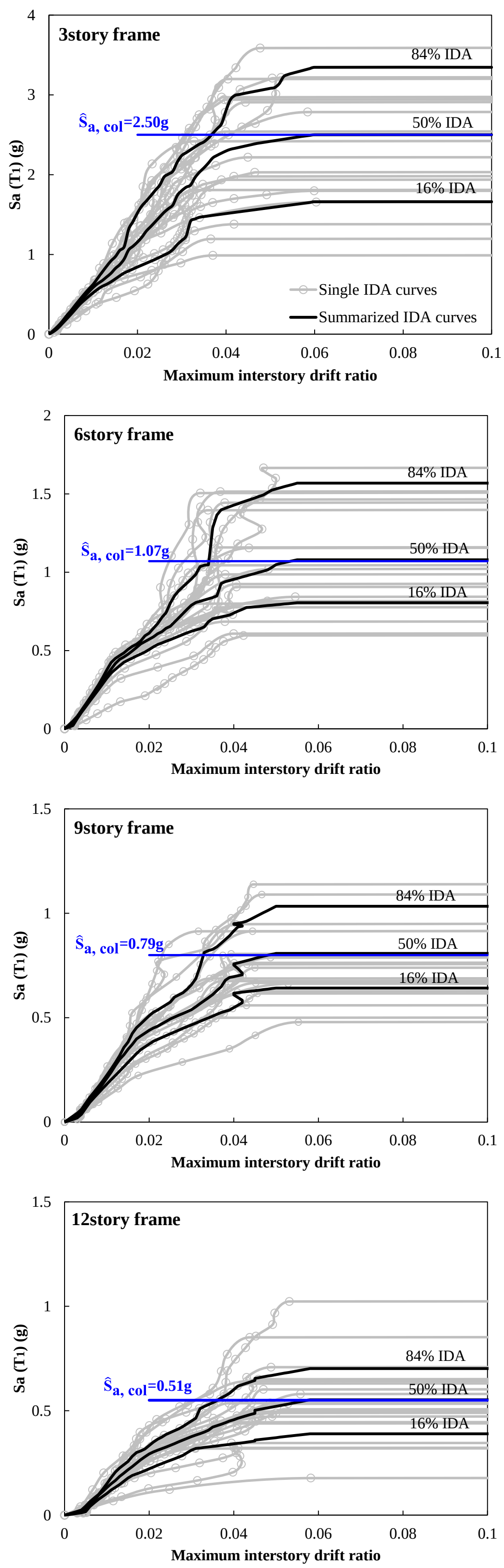


Fig. 5 IDA results for the example structures subjected to the set of twenty ground motion records when the maximum interstory drift ratio is considered as the EDP

also shown in the same figures. It is noted that IDA results will be used as the benchmark solution for the evaluation of

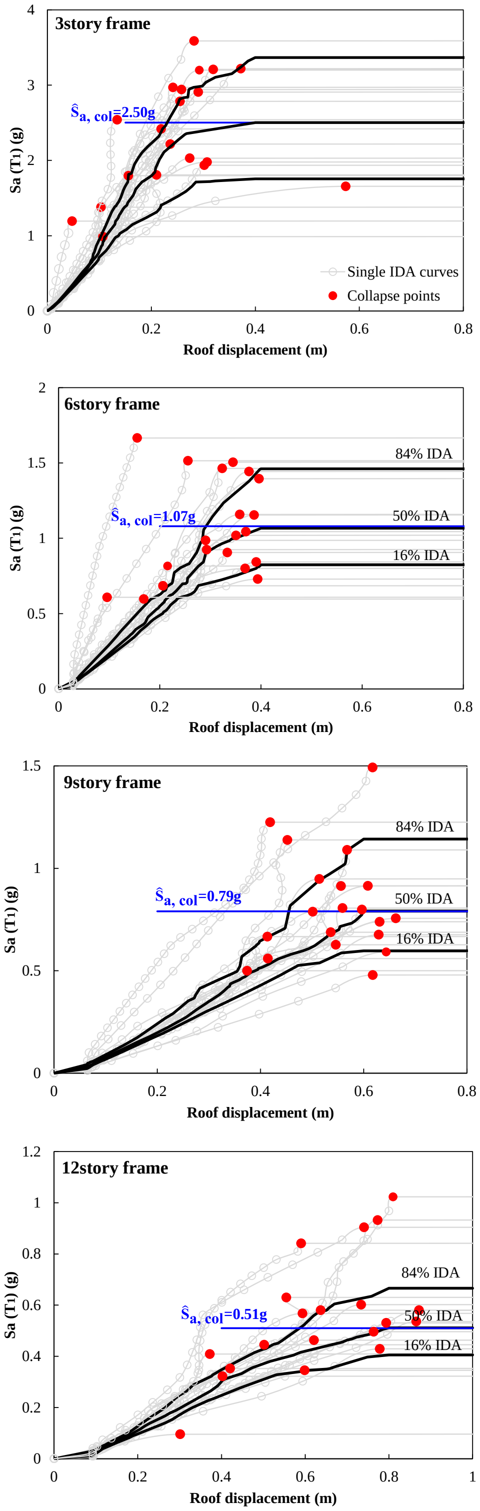


Fig. 6 IDA results for the example structures subjected to the set of twenty ground motion records when the roof displacement is considered as the EDP. The collapse points are shown by filled circles

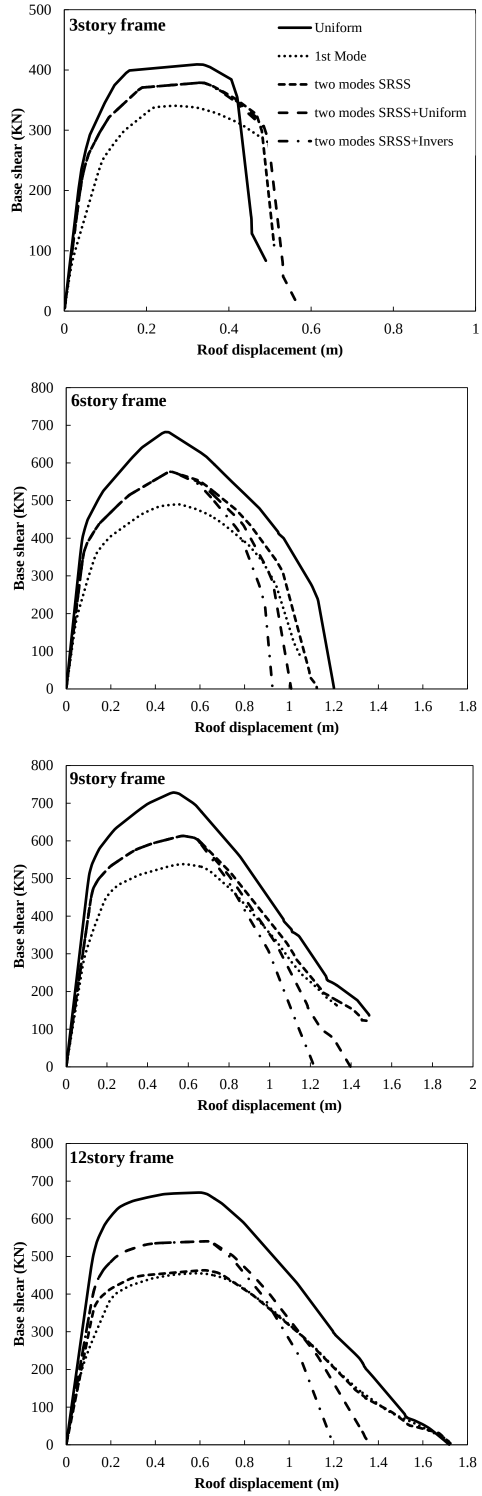


Fig. 7 Pushover curves for the studied RC frames produced by four different lateral load patterns

simplified nonlinear analysis methods in the subsequent sections.

4.2 Results of simplified nonlinear analysis procedures

4.2.1 SPO2IDA method

The SPO2IDA method benefits from an Excel-based tool to convert the pushover curve of a first-mode-dominated MDOF structure into the summarized 16%, 50% and 84% IDA curves by using the nonlinear time-history analysis results of numerous SDOF systems. It requires using an equivalent SDOF oscillator whose backbone curve closely matches the pushover curve of the original MDOF structure. Because the pushover curve for a MDOF structure cannot uniquely be defined, SPO2IDA method recommends performing several nonlinear static analyses with different lateral load patterns to identify the most damaging and least-energy (i.e. the worst-case) pushover curve that leads to global collapse. In SPO2IDA method, an equivalent SDOF system whose backbone curve mimics such a pushover curve can predict the dynamic response of the real MDOF structure with more accuracy. In the present research, based on the recommendations by Vamvatsikos and Cornel (2005) several "single-stage" and "two-stage" nonlinear static analyses are performed to obtain the worst-case pushover curve for each of the example structures. Three different lateral load distributions are used for performing the single-stage pushover analyses including distributions proportional to the story masses (i.e., uniform), the first elastic mode shape, square-root-of-sum-of-squares (SRSS) combination of the first two elastic mode shapes. However, two-stage pushover analyses benefit from consecutive implementation of single-stage ones. In the first stage, a nonlinear static analysis is performed by using the load pattern proportional to the SRSS combination of the first two elastic mode shapes until the base shear of the building reaches its peak value. Then, the second stage of the analysis is continued by changing the load pattern to the uniform or the inverse of the pre-peak SRSS one.

Fig. 7 shows the pushover curves for the example structures subjected to each of the selected load patterns. As shown in Fig. 7, the first-mode lateral load pattern produces the worst-case pushover curve in most cases and will be used for SPO2IDA analysis in this study.

Fig. 8 shows the 16%, 50%, and 84% fractile IDA curves estimated by SPO2IDA approximate procedure along with those given by the exact nonlinear response history analysis for the 3-, 6-, 9-, and 12-story buildings. The median seismic collapse capacities obtained from SPO2IDA method are also shown in the same figure. A comparison between the seismic capacities obtained from SPO2IDA and IDA methods for three different performance levels (i.e., life safety (LS), collapse prevention (CP) and global instability (GI)) are also provided in Table 4. These performance levels can be obtained from IDA curves. In the present study, LS performance level is assumed as the limit state in which the structure exceeds 2% MIDR, CP is reached when the local slope on the IDA curve is 20% of the elastic slope or the MIDR is equal to 10%, whichever occurs first. GI is also evident by the unbound increase of EDP in each IDA curve, where the curve becomes flat.

As can be seen in Fig. 8, the approximate curves generally agree with those given by the exact IDA approach; however, the level of agreement varies from one structure to another and at different ranges of roof displacement. More specifically some differences can be seen immediately after the linear elastic region, but they are reduced at higher displacement values. There is a good correlation between the results near the collapse region. This is especially true for the summarized 50% fractile IDA curves. Comparison of estimated collapse capacities with the exact values shows that SPO2IDA method can predict

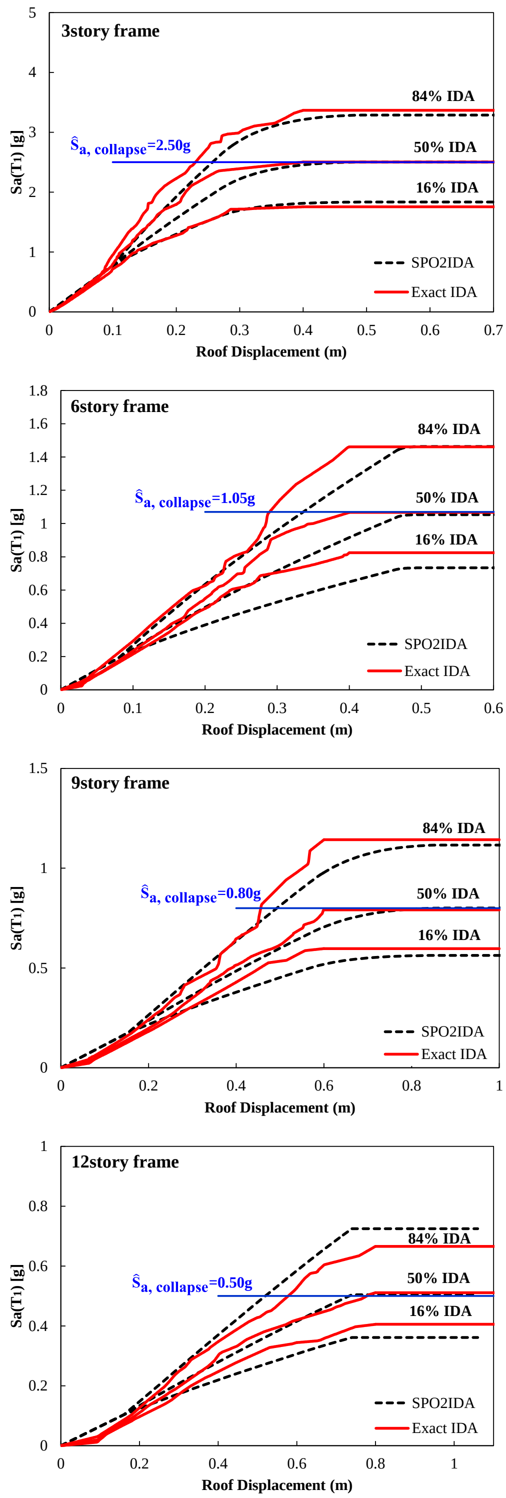


Fig. 8 Sixteen, 50 and 84% fractile IDA curves for the 3-, 6-, 9- and 12-story example structures from nonlinear RHA (exact) versus SPO2IDA approximate procedure

the median collapse capacity of the example structures fairly well. However, less accurate predictions from SPO2IDA method for the 16% and 84% fractile IDA curves are obtained for the collapse prevention (CP) and global instability (GI) limit states (see Table 4). The exact values obtained as median collapse capacity for the 3-, 6-, 9- and

Table 4 Comparison of approximate seismic capacities ($S_a(T_1)$) obtained from SPO2IDA with those given by the exact IDA method for three different performance levels

Building	Limit state	16% (g)		50% (g)		84% (g)	
		IDA	SPO2IDA	IDA	SPO2IDA	IDA	SPO2IDA
3story	LS	0.84	0.87	1.16	1.05	1.52	1.31
	CP	1.47	1.46	2.25	1.90	2.96	2.69
	GI	1.76	1.82	2.50	2.50	3.37	3.29
6story	LS	0.51	0.40	0.57	0.50	0.61	0.60
	CP	0.70	0.51	1.05	0.90	1.05	0.92
	GI	0.82	0.73	1.07	1.05	1.46	1.46
9story	LS	0.37	0.34	0.44	0.41	0.51	0.57
	CP	0.58	0.49	0.69	0.64	0.93	0.82
	GI	0.60	0.56	0.79	0.80	1.14	1.12
12story*	LS	0.22	0.20	0.29	0.28	0.33	0.35
	CP & GI	0.39	0.36	0.55	0.50	0.70	0.72

*12-story frame reaches global instability quite early, so GI and CP limit states coincide.

12-story frames from the 50% fractile IDA curves (see Figs. 5 and 6) are equal to 2.50 g, 1.07 g, 0.79 g and 0.51 g, respectively; i.e., the estimation errors are generally less than 2% for these buildings.

4.2.2 MPA-based IDA method

In this section, the seismic collapse response of the buildings are evaluated by the approximate MPA-based IDA method and compared with the exact IDA results. Fig. 9 shows the IDA curves for the first-mode SDOF system of the structures subjected to the selected ground motion records. The summarized IDA curves and the median collapse capacity predicted by the MPA-based IDA for each building are also shown in the same figure.

As can be seen in Fig. 9, MPA-based IDA method can estimate the median collapse capacity and the 16%, 50% and 84% fractile IDA curves fairly well. The median collapse capacities predicted by the MPA-based IDA are 2.51 g, 1.03 g, 0.81 g and 0.51 g for the 3-, 6-, 9-, and 12-story buildings, respectively; i.e., the estimation errors are less than 0.5%, 4%, 2.5% and 0.5% for these structures, respectively. However, similar to SPO2IDA, less accurate collapse responses are obtained for the 16% and 84% fractile IDA curves; such that the accuracy of the method decreases as the number of stories is increased. For example, in the 12-story frame, the collapse capacities for the 16% and 84% fractile IDA curves are underestimated by 17% and 10% compared to the exact IDA results, respectively; whereas the estimation errors for the 3-story building are almost 3% and 4%, respectively. In MPA-based IDA analyses, the first-mode participation factor is used as the transformation factor to change the SDOF system to MDOF one for the example buildings.

Table 5 also summarizes the seismic capacities estimated by MPA-based IDA for LS, CP and GI limit states and compares them with the exact IDA results. As shown in Table 5, less accurate seismic resistant capacities for LS and

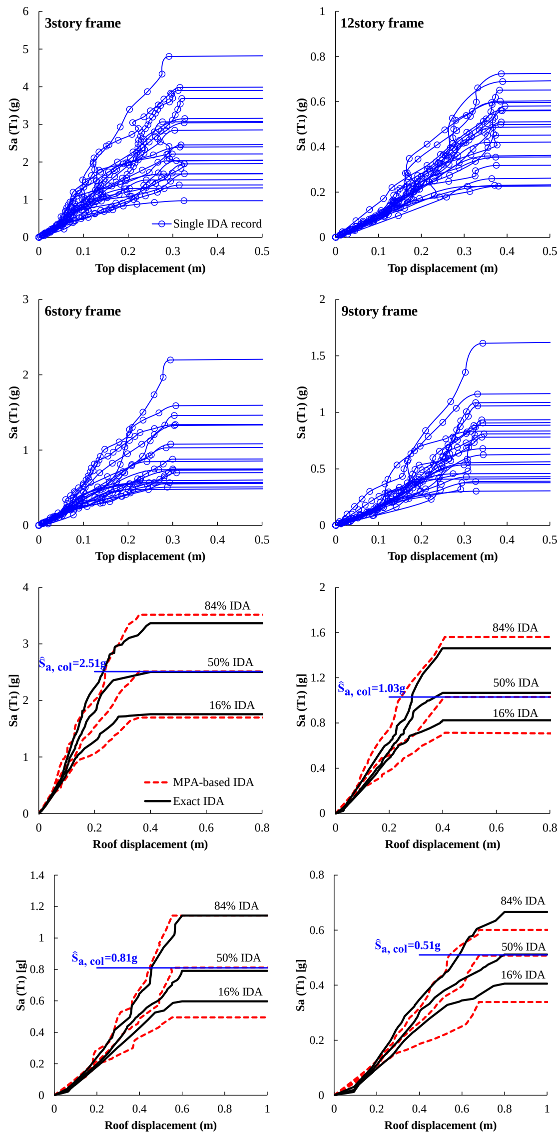


Fig. 9 IDA curves for the first-mode SDOF systems and the corresponding MPA-based IDA results for the example buildings

CP limit states are obtained by the MPA-based IDA. Nonetheless, the performance of the method in estimating the 84% fractile IDA curves appears to be better than SPO2IDA procedure.

4.2.3 Shafei et al. method

Here, the median seismic collapse capacity of the reference structures are determined by the Shafei *et al*. method and compared with the exact IDA results. The procedure estimates the median seismic collapse capacity of shear-wall and moment-resisting frame structures by using some closed-form equations. Nonetheless, because the procedure doesn't provide all important fractiles of IDA curves, it fails to fully evaluate the seismic collapse response of these buildings. A trilinear representation of the pushover curves, as recommended by ASCE-41 (2013), are used for extracting the required parameters for each building.

Table 5 Comparison of approximate seismic capacities ($S_a(T_1)$) obtained from MPA-based IDA with those given by the exact IDA method for three different performance levels

Building	Limit state	16% (g) IDA	16% (g) MPA-based IDA	50% (g) IDA	50% (g) MPA-based IDA	84% (g) IDA	84% (g) MPA-based IDA
3story	LS	0.84	0.77	1.16	1.02	1.52	1.65
	CP	1.47	1.20	2.25	1.80	2.96	2.95
	GI	1.76	1.70	2.50	2.51	3.37	3.52
6story	LS	0.51	0.39	0.57	0.56	0.61	0.72
	CP	0.70	0.54	1.05	0.97	1.05	1.17
	GI	0.82	0.70	1.07	1.03	1.46	1.56
9story	LS	0.37	0.29	0.44	0.42	0.51	0.55
	CP	0.58	0.49	0.69	0.76	0.93	1.01
	GI	0.60	0.50	0.79	0.81	1.14	1.14
12story*	LS	0.22	0.17	0.29	0.26	0.33	0.30
	CP & GI	0.39	0.34	0.55	0.51	0.70	0.67

*12-story frame reaches global instability quite early, so GI and CP limit states coincide.

Table 6 Parameters used for the estimation of median seismic collapse capacity of the reference structures by using the Shafei *et al*. method; errors relative to the exact IDA are also presented

Frame	θ_{pl}	θ_{pc}	α_T	γ	z_0	z_1	z_2	z_3	z_4	$\hat{\eta}_c(g)$	error (%)
3story	0.30	0.32	0.21	1.68	-1.74	0.160	1.51	0.022	0.027	1.40	-44
6story	0.22	0.42	0.17	2.04	-3.42	0.058	2.26	0.030	0.021	1.05	-2.0
9story	0.15	0.50	0.16	2.33	-4.81	0.045	2.95	0.020	0.018	0.72	-8.0
12story	0.13	0.56	0.16	2.49	-5.61	0.028	3.56	0.019	0.013	0.51	+0.1

*Positive error indicates Shafei *et al*. method overestimates the median collapse capacity relative to the exact IDA method.

Table 6 reports the parameters extracted from the idealized pushover curve and the median seismic collapse capacity estimated by the method for each building. The errors relative to the exact IDA approach are also presented in the same table for comparison. The results show that, except the 3-story building, Shafei *et al*. method can estimate the median sidesway collapse capacity of the example structures with sufficient accuracy. Nonetheless, less accurate results are obtained by the method for some case studies compared to SPO2IDA and MPA-based IDA methods.

The main advantage of the Shafei *et al*. method is that it directly uses MDOF models for estimating the median seismic collapse capacity of structures. These models can predict more realistic results for different global or local collapse modes as they can to some extent take into account the effect of cyclic deterioration in strength and stiffness of structural components through the nonlinear analysis. Furthermore, because the Shafei *et al*. method uses some simple closed-form equations for the estimation of seismic collapse capacity, it needs much less computational effort. As it can be seen in Table 6, the errors from the Shafei *et al*. method are less than 2%, 8% and 0.5% (corresponding to 1.05 g, 0.72 g and 0.51 g median sidesway collapse capacities) for the 6-, 9- and 12-story buildings, respectively. However, less accurate collapse capacity is obtained for the 3-story frame (i.e., 44% error). This deficiency may be attributed to the fact that the z_i factors are not well defined by the procedure for structures with the number of stories less than 4 (See Table 1).

Table 7 Parameters used for the estimation of median seismic collapse capacity of the reference structures by using the Hamidia *et al*. method; errors relative to the exact IDA are also presented

General properties				Hamidia *et al*. method							Modified Hamidia *et al*. method					
Frame	T_{el}	$\phi_{I,r}$	Γ_I	A_y^* (g)	δ_u (m)	μ_T	r	CMR	$\hat{S}_{a,col}$ (g)	Error (%)	δ_u (m)	μ_T	r	CMR	$\hat{S}_{a,col}$ (g)	Error (%)
3story	0.62	0.51	2.67	0.51	0.51	7.71	6.10	2.24	3.10	+24	0.30	4.55	4.12	1.51	2.09	-16
6story	1.01	0.78	1.64	0.37	0.79	6.54	5.92	2.56	2.18	+103	0.34	2.83	2.88	1.25	1.06	-1
9story	1.47	0.86	1.48	0.22	0.86	5.63	5.77	2.21	1.29	+64	0.55	3.59	3.78	1.45	0.85	+7
12story	1.88	0.90	1.44	0.13	0.90	6.11	5.44	1.55	0.71	+38	0.62	4.22	4.11	1.17	0.53	+5

*A_y is the yield strength of the first-mode SDOF model.

4.2.4 Hamidia et al. (2013) method

This section reports the median seismic collapse capacity of the buildings obtained from Hamidia *et al*. method. The procedure uses the response of nonlinear static analysis to estimate the CMR and the median seismic collapse capacity of frame structures against strong earthquakes. In this paper, each of structural models is first subjected to the first-mode nonlinear static analysis until a loss of 20% in the base shear capacity is reached. This displacement will then be used as the ultimate roof displacement (δ_u) for idealizing the pushover curve and computing the target ductility demand (μ_T) (see dashed curves in Fig. 10). The idealization is performed using FEMA-P695 recommendations. Table 7 summarizes the parameters required by the method for computing the CMR and median collapse capacity for each building. The estimation errors relative to the exact IDA approach are also presented in the same table for comparison.

The results illustrate that the procedure leads to less accurate median seismic collapse capacity of the buildings compared to those given by the other simplified nonlinear analysis methods. This may be attributed to the characteristics of the idealized pushover curve. In an attempt to further improve Hamidia *et al*. method two aspects are considered in the following, namely the definition of the collapse point and the definition of the yield strength.

First, the median ultimate roof displacement predicted by IDA is considered as the ultimate displacement and the procedure is re-evaluated. The value of the new target-ductility demands are shown in Fig 10. The results are also reported in Table 7. As it can be seen from this Table, except for the 3-story frame, the results show a good correlation between the modified Hamidia *et al*. method with those given by the IDA. The median collapse capacities equal to 2.09 g, 1.06 g, 0.85 g and 0.53 g are

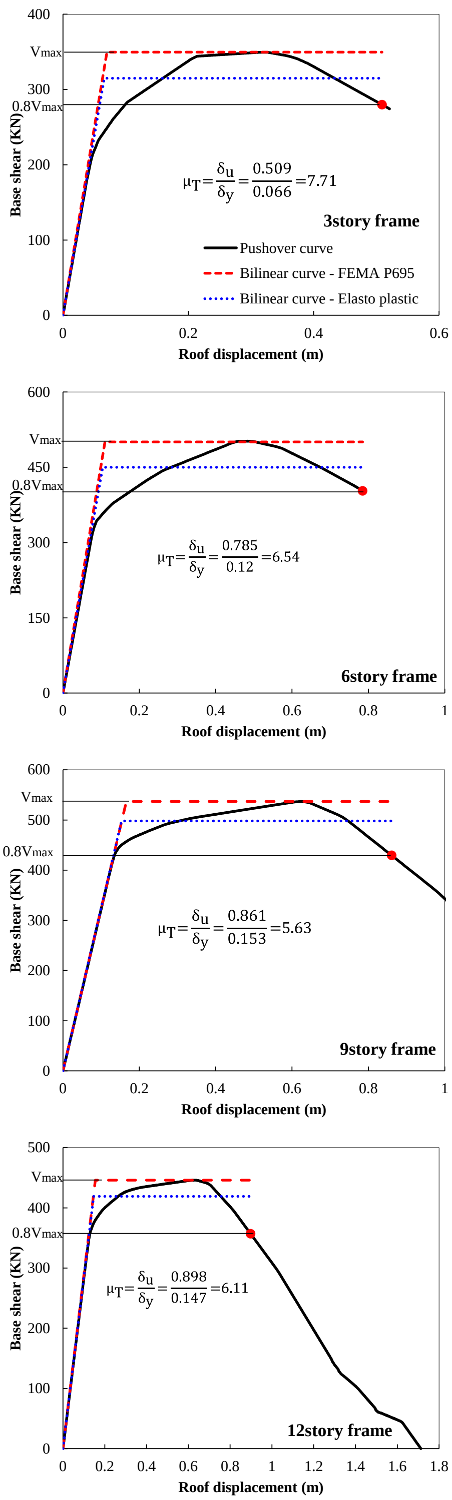


Fig. 10 Bilinear representation of pushover curves based on EPP and FEMA-P695. Target ductility demands required by the Hamidia *et al*. method are also shown in the plots

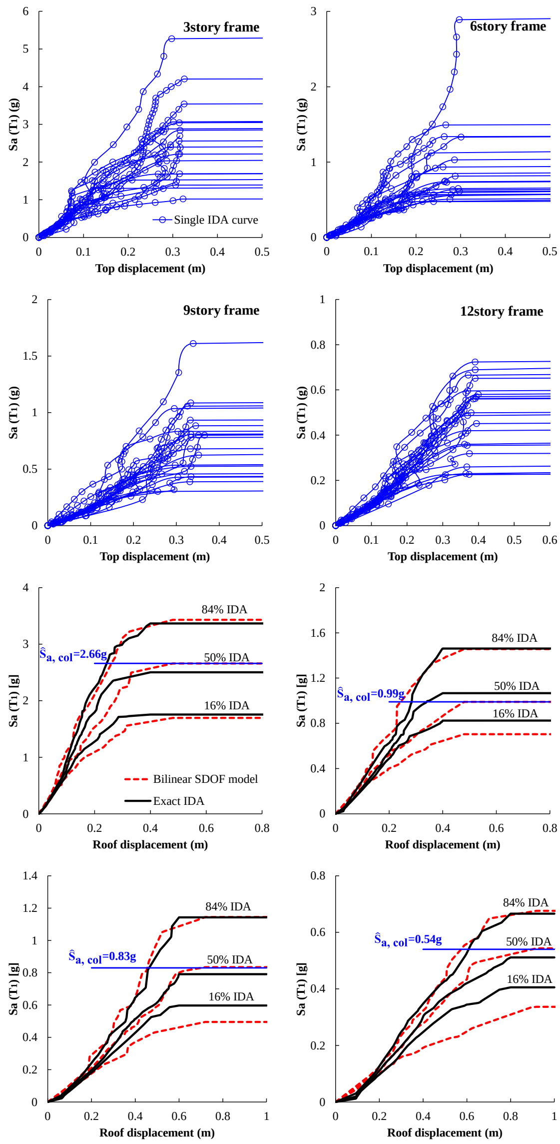


Fig. 11 IDA curves for the equivalent SDOF systems with backbone curves displayed in Fig. 10 (dotted curves), and the corresponding IDA results for the example buildings. The median collapse capacities are also shown in the plots

obtained for the 3-, 6-, 9-, and 12-story buildings, respectively; i.e., the estimation errors are less than 16%, 1%, 7% and 5% for these structures, respectively. Accordingly, it is concluded that the ultimate roof displacement assumed by the Hamidia *et al*. method may not be sufficiently accurate for RC buildings with medium ductility.

Second, to investigate the effect of the defined yield strength, an inelastic equivalent SDOF model with bilinear Elastic Perfectly Plastic (EPP) backbone curve is established and subjected to IDAs. This idealization is similar to the one that has been used by FEMA-P695 with the exception of applying equal absorbed energy for the original and idealized curves up to the collapse point. Here a variant of the FEMA-P695 method is attempted as it was found to be a simple and practical one (see dotted curves in Fig. 10).

Fig. 11 shows the IDA curves of the first-mode equivalent SDOF oscillator for each building with the selected bilinear backbone curve. The summarized 16%, 50% and 84% IDA curves and the values of median collapse capacity for each building are also shown in Fig. 11. As it can be seen from Fig. 11, using the proposed bilinear representation of the pushover curve, the seismic collapse response of structures can be estimated with acceptable level of accuracy. The median collapse capacities equal to 2.66 g, 0.99 g, 0.83 g and 0.54 g are obtained for the 3-, 6-, 9-, and 12-story buildings, respectively; i.e., the estimation errors are about 6%, 7%, 5% and 6% for these buildings, respectively.

The above brief evaluations show that bilinear idealization used by Hamidia *et al*. may need to be adjusted for RC structures. The second alternative considered above may be considered as a possible practical modification which is also capable of producing summarized IDA curves. Although a good correlation is observed between the obtained summarized 50% and 84% IDA curves with those given by the exact IDA method, but like to SPO2IDA and MPA-based IDA methods, less accurate results are obtained for the 16% fractile IDA curves.

Herein a simplified method is used for collapse response assessment of regular RC moment-resisting frames. Because the method uses IDA results of an equivalent SDOF system, it can also be applied to other types of buildings with different structural systems. Nevertheless, the procedure cannot take into account the effect of cyclic strength and stiffness degradation of structural components, as well as the structural collapse modes dominated by forces or overturning moments. To investigate the accuracy and effectiveness of the proposed method for seismic collapse capacity prediction of other types of structural systems additional studies may be required.

5. Conclusions

The issue of seismic collapse assessment for RC frame structures was studied in this paper considering simplified methods based on nonlinear static analysis. A review of different available methods was carried out and then four methods were chosen for a more detailed assessment of their accuracy and efficiency. Four RC intermediate moment-resisting frames with 3-, 6-, 9- and 12-stories, designed based on current US building codes, and a set of twenty far-field ground motion records were used for this assessment. The performance of the simplified methods was evaluated by comparing the calculated median collapse capacities with those given by the exact IDA method. The accuracy of SPO2IDA and MPA-based IDA methods in approximating the summarized (16%, 50% and 84% fractile) IDA curves and structural capacities for three different limit states (LS, CP and GI), was also investigated. Based on the results of various pushover and incremental dynamic analyses carried out for RC structures with different heights the main findings of the study are summarized as follows:

- The assessment of the results obtained from this study indicates that, except the method proposed by Hamidia *et al*. the other simplified analysis methods can reliably be used for estimating the median seismic collapse capacity of regular RC moment-resisting frames. Some methods use recommended closed-form equations to estimate the collapse capacity but others also provide for the IDA curves. Generally, the methods which produce IDA curves fail to approximate the summarized 16% and 84% fractile IDA curves with sufficient accuracy.
- SPO2IDA predicts sufficiently accurate results for the median seismic collapse capacity of the regular RC frames. The estimation errors are less than 2% in the all example buildings. The method is also capable of predicting the seismic capacities for different limit states (LS, CP and GI) with errors less than 9%. It was also noted that the accuracy of SPO2IDA is strongly dependent on the lateral load pattern selected for the pushover analysis. More accurate results typically correspond to the worst-case (least-energy) pushover curve of the structure. Therefore, for the efficient application of the method suitable lateral load pattern should be considered.
- Although MPA-based IDA requires a small fraction of the computational time compared to that required in the exact IDA method, but it is still considered as a computationally-demanding procedure compared to other simplified methods studied in this paper. Like to SPO2IDA, MPA-based IDA also provides fairly accurate estimates of structural capacities for different limit states (LS, CP and GI) in most case studies. A reasonable approximation of summarized 50% fractile IDA curves is achieved by the method for the reference buildings. Nonetheless, the accuracy of the method deteriorates in approximating the 16% and 84% fractile curves for the 12-story frame whose dynamic response is complex due to the significant higher mode effects. Compared to SPO2IDA, the summarized IDA curves approximated by MPA-based IDA are much closer to those given by the exact IDA method, in most cases.
- The method proposed by Shafei *et al*. produces sufficiently acceptable results for the median collapse capacity of the analyzed buildings except in the case of the low-rise 3-story frame, where poor estimates are obtained. This shortcoming is attributed to the fact that the z_i factors (see Table 1) are not well defined by the procedure for structures with the number of stories less than four. Among the simplified methods studied in this paper, this method is identified as the simplest procedure which can reliably estimate the median sidesway collapse capacity of mid- to high-rise frame buildings with the minimum computational efforts.
- The method proposed by Hamidia *et al*. results in poor estimates for the seismic collapse capacity of the example buildings. This is possibly due to the assumed characteristics of the idealized pushover curve. According to the IDA results, it is demonstrated that more accurate results can be obtained by the proposed method if the ultimate roof displacement is estimated with sufficient accuracy. Also, it is shown that an EPP bilinear representation of the pushover curve may be suitable for developing the backbone curve of the equivalent SDOF model.

Finally, based on the results presented in this study, the method proposed by Shafei *et al*. (2011) can confidently be used as a rapid analysis tool for estimating the median seismic collapse capacity of RC frames. The SPO2IDA and MPA-based IDA methods are also capable of producing reliable estimates of the seismic demand and capacity of structures from low to high levels of ground motion intensity with sufficient accuracy. Accordingly, these methods are more suitable for a comprehensive seismic collapse response assessment of RC frames.

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