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A simplified normalized multi-mode nonlinear static procedure (NMP) for seismic performance evaluation of building structures

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Abstract

The current work presents a simplified multi-mode nonlinear static procedure based on normalizing the deformation demands (called NMP) for estimating the seismic demands of structures with significant higher mode effects. The proposed procedure is conceptually based on the extended N2 (where N stands for nonlinear analysis and 2 for two mathematical models) method (Kreslin and Fajfar in Earthquake Engineering & Structural Dynamics 40:1571–89, 2011). However, the modal combination procedure used in the proposed method to take the higher mode effects into account is different. The NMP procedure generally encompasses two major steps. First, the structure is pushed using a conventional pushover procedure, namely the basic N2 method, the first-mode pushover analysis, or a pushover analysis with a triangular or a uniform load pattern, which mostly controls the responses at the lower part of the structures. Second, the modal story displacements and drifts computed based on the modal response analysis concept for all the modes of interest are algebraically added. These combined responses are normalized based on the predetermined target displacement. The final responses are the envelope of the results obtained from the two aforementioned steps. The approach used in the second step can preserve the signs of modal responses. Therefore, the effects of sign reversal due to the contribution of higher modes are included in the proposed method. Moreover, the NMP procedure mitigates the computational burden compared to nonlinear response history analysis or adaptive pushover procedures. In order to evaluate the accuracy of the proposed procedure in estimating the seismic demands of structures, it is applied to two sets of steel structures including three special moment frames (SMFs) with 6-, 12-, and 18-story heights, and a 12-story SMF with soft stories as the first set and the 9-, and 20-story SAC steel moment frames as the second set. Comparing the results of the NMP procedure with those obtained by the nonlinear response history analysis (NL-RHA) as the benchmark solution and two other pushover approaches, including a conventional first-mode-based pushover analysis and the extended N2 method, demonstrate the sufficiency of the NMP procedure in accurately estimating the global and local response quantities.

Keywords Nonlinear static analysis · Extended N2 pushover analysis · Higher mode effects · Seismic evaluation · Performance assessment · ASCE 41

Extended author information available on the last page of the article

1 Introduction

Despite the outstanding rigor of the nonlinear response history analysis (NL-RHA) in estimating the seismic performance of structures, its conceptual and numerical intricacy, time-consuming process, and the difficulty encountered in appropriately selecting and scaling ground motion records restrict its use in practice. These problems have caused the nonlinear static procedures (NSPs) or pushover methods to gain immense popularity among practitioners and researchers as a suitable approach for seismic design and performance evaluation purposes of structures. That is, the pushover procedures are widely used in engineering offices all over the world because they can provide better insight into structural deficiencies, more information about the response characteristics, and the lateral loading capacity of the structure subjected to seismic loads compared to the linear procedures. However, the earlier versions of the NSPs are limited to the fundamental mode of structure and cannot identify possible failure mechanisms due to the higher modes effect (Krawinkler 1998; Fajfar 2000; Chopra and Goel 2002). Further, conventional pushover procedures neglect the variations in the dynamic characteristics of structure for computing lateral load pattern, which leads to a consistently invariant lateral load distribution during the analysis (Bracci et al. 1997; Elnashai 2001; Mwafy and Elnashai 2001; Antoniou and Pinho 2004a, b; Kalkan and Kunnath 2006). The aforementioned drawbacks of conventional pushover procedures were a motivation for researchers to develop enhanced pushover procedures, thereby, to some extent, alleviating these inherent shortcomings. In this regard, several attempts have been made in recent years to the development of enhanced NSPs.

The modal pushover analysis (MPA) (Chopra and Goel 2002) based on the structural dynamics theory is a pioneer of multi-mode pushover procedures that can somewhat overcome the shortcomings of conventional NSPs. However, the use of a quadratic modal combination rule (for example the square-root-of-sum-of-squares (SRSS) or the complete quadratic combination (CQC) rules) in this method to combine the peak modal responses often results in an overestimation of the ultimate results (Kim and Kurama 2008). Inaccuracy in the estimation of the local demands, particularly the member forces and plastic hinge rotations, is another deficiency of the MPA procedure (Chopra and Goel 2002; Goel and Chopra 2005; Poursha et al. 2008, 2009; Fragiadakis et al. 2014). Moreover, the MPA procedure requires nonlinear response history analysis of the equivalent SDOF systems to compute the peak roof displacements corresponding to each mode of interest. This process is a laborious task and is not as computationally attractive as some other methods. The modified MPA (MMPA) (Chopra et al. 2004) procedure is the improved version of the MPA based on the assumption that the structure remains elastic when vibrating in higher modes. Although the MMPA procedure reduces the computational effort compared to its original version, it cannot remedy the conservatism of the MPA, and it results in a large estimation of seismic demands, especially for lightly damped systems with a damping ratio considerably less than 5% (Chopra et al. 2004). The MPA was also extended to asymmetric-plan buildings under one component of earthquake ground motion (Chopra and Goel 2004) and three-dimensional symmetric- or asymmetric-plan buildings subjected to bi-directional earthquake excitations (Reyes and Chopra 2011).

Jan et al. (2004) proposed the upper-bound (UB) pushover procedure wherein the upper-bound of contribution ratio of the second mode in determining the lateral load pattern and target roof displacement is utilized. This method substantially underestimates the seismic demands at lower stories and overestimates them at upper levels. For this reason, the UB procedure was then improved by proposing the modified upper-bound (MUB) and the

extended upper-bound (EUB) pushover procedures by Poursha and Talebi Samarin (Poursha and Samarin 2015) for symmetric-plan and one-way asymmetric-plan tall buildings, respectively, and the improved upper-bound (IUB) (Rahmani et al. 2017) and the adaptive upper-bound (AUB) methods by Rahmani et al. (Rahmani et al. 2019). In another research, the mass proportional pushover analysis (MPP) was developed by Kim and Kurama (Kim and Kurama 2008). In this method, the contribution of higher modes to response is accumulated in a single invariant lateral load pattern, which is proportional to total seismic masses at the floors and roof levels. Therefore, only one pushover analysis is conducted without using any modal combination rule. Inspired by the MPA method, Poursha et al. (Poursha et al. 2009) proposed the consecutive modal pushover (CMP) procedure in which the MPA procedure is applied consecutively in two or three stages in a single pushover analysis with an invariant force distribution. In this approach, the final responses are calculated by enveloping the obtained results from a single-stage pushover analysis using a triangular or uniform load pattern and two or three multi-stage pushover analyses by means of the consecutive implementation of the modal pushover analysis. A disadvantage of this procedure is the convergence problems encountered during the multi-stage analysis when one stage finishes and the other starts. Most recently, the updated version of the consecutive modal pushover (UCMP) procedure was also proposed by Zarrin et al. (Zarrin et al. 2021) for estimating the ductility level earthquake design demands of jacket offshore platforms. In the UCMP method, the changes in dynamic characteristics of the structure are taken into account in determining the applied load distribution while the non-adaptive nature of the CMP method is retained.

Poursha and Amini (Poursha and Amini 2015) developed a single-run multi-mode pushover (SMP) analysis procedure wherein the effect of higher modes was taken into account using a single invariant lateral force distribution obtained from algebraically adding the modal story forces. The frequency content of a given input ground motion and the effect of sign inversion in the story forces of higher modes are reflected in the applied multi-mode lateral load pattern. In another investigation, Behnamfar et al. (Behnamfar et al. 2016) introduced a displacement pushover procedure based on the spectral analysis method called the extended drift pushover analysis procedure (EDPA). Simple summation of modal responses modified using modal correction factors is used in this method instead of the SRSS or CQC rules to maintain the sign of modal responses in computing the lateral load pattern. The accuracy of this procedure was not checked for story drift demands. Later on, a non-adaptive displacement-based pushover (NADP) procedure was proposed by Amini and Poursha (Amini and Poursha 2016) in which the envelope of the desired results obtained by a conventional, and one or two single-run displacement-based pushover analyses with invariant load patterns is taken as the ultimate seismic demand of the structure. On one hand, the NADP procedure yields a satisfactory estimation of structural responses in higher story levels, but on the other hand, the method suffers a serious disadvantage due to the underestimation of seismic demands in lower floors. Liu and Kuang (Liu and Kuang 2017) devised a simple non-adaptive nonlinear static procedure named spectrum-based pushover analysis (SPA) for estimating seismic demands of tall buildings, whereby the coupling effect of different modes is considered using a simplified approach. In this method, the lateral load vectors determined based on the elastic mode shapes of the structure are applied consecutively to the structure until a predefined roof displacement computed based on the response spectrum analysis method is reached. The SPA procedure was also extended for several other buildings (Liu et al. 2018a, b, 2020). Guan et. al. (Guan et al. 2019) suggested a simple evaluation method to consider the higher mode effects through the combination of the results obtained by three conventional pushover procedures

with different loading patterns including uniformly distributed loading, first-mode vibration loading, and concentrated loading at the vertex. Despite the simplicity of this combination model, it is merely applicable to concrete-filled steel tubular (RCFT) buildings and the interstory drift is the only response parameter that this model is formulated for. Habibi et al. (Habibi, Saffari, and Izadpanah 2019) put forward a method in which an optimal lateral load pattern (OLLP) for pushover analysis with the best compatibility to the nonlinear dynamic procedure is attained using an optimization procedure. The complete dependence of this approach on nonlinear dynamic analysis is the major demerit of this procedure, which considerably declines the conceptual simplicity and computational attractiveness of the pushover procedure and consequently makes it undesirable for practitioners.

Apart from the above-referenced pushover procedures with invariant load distributions, multiple adaptive multi-mode pushover procedures (Bracci et al. 1997; Gupta and Kunnath 2000; Elnashai 2001; Antoniou and Pinho 2004a; b; Kalkan and Kunnath 2006; Shakeri et al. 2010; Abbasnia et al. 2013; Amini and Poursha 2018; Jalilkhani et al. 2020) were also proposed to take into account the changes in the dynamic characteristics of the structure due to progressive stiffness degradation of components and members yielding. Therefore, an eigenvalue analysis is required at the start of each loading increment to update the load pattern, which leads to a more computational burden. However, the additional computational effort imposed by the adaptive solution cannot guarantee to result in a superior estimation of seismic demands compared to non-adaptive approaches. Take, for example, the FAP procedure (Antoniou and Pinho 2004a), which even tends to yield worse results compared to conventional pushover procedures (Daei and Poursha 2021). Moreover, any engineering software used in practice does not have the possibility to implement an adaptive algorithm in it. In literature, there are also some enhanced pushover procedures (Sucuoğlu and Günay 2011; Surmeli and Yuksel 2015; Brozovič and Dolšek 2014) with a rather convoluted algorithm. It is noteworthy that the simplicity of the NSP is the reason for the popularity of this procedure and if a simplified analysis procedure becomes overly complicated, it will probably fail to gain any interest in the civil engineering profession (Kreslin and Fajfar 2011).

One of the world-renowned and theoretically robust enhanced pushover procedures is the extended N2 procedure proposed by Kreslin and Fajfar (Kreslin and Fajfar 2011, 2012) to take into account the higher mode effects in plan and/or elevation. This method presumes that the building remains elastic when vibrating in higher modes. To implement this premise and take the higher mode effects into consideration, elastic modal analysis is used. The extended N2 procedure can provide a satisfactory estimation of seismic demands. However, changes in the sign of the story displacements for the higher modes are suppressed in this procedure because of using the quadratic modal combination rules in the elastic modal analysis. Further, the accuracy of the extended N2 method in estimating plastic rotations of elements was not appraised.

To somehow overcome these limitations inherent in the extended N2 method, this paper aims to propose an alternative for the extended N2 procedure. The proposed pushover method, called the normalized multi-mode pushover (NMP) procedure, is as practical and less onerous as its counterpart. As with the extended N2 method, in the proposed NMP procedure, the ultimate seismic demands are computed by enveloping the results obtained from the conventional pushover analysis and the elastic modal analysis. The major difference between the proposed procedure and the extended N2 procedure is in the combination rule of the modal responses. In the NMP method, the peak story displacements and drifts in the second step are calculated by the algebraic sum of the modal responses derived using the modal response analysis concept. This is in contrast to the quadratic modal combination rule employed by the extended N2

method. The use of the direct summation method for combining the modal responses in the NMP method helps to maintain the sign inversion due to the contribution of higher modes. Moreover, one of the promising features of the proposed procedure is its less computationally demanding process compared to the majority of enhanced nonlinear static procedures in literature; such that the second step of the method requires no analysis, and the results are taken by using the given equations. The basic idea of the proposed procedure bears a close resemblance, with some crucial modifications, to the multi-mode N2 (MN2) pushover procedure recently proposed by (Mohamad Zarrin et al. 2020a, b) for ductility level seismic performance evaluation of jacket-type offshore platforms. The main difference between the proposed procedure and the MN2 method is that two modal combination rules are utilized herein using both the absolute value of the modal participating factor, disregarding its sign, and its original value for computing the drifts, while only the original value of the modal participating factor was utilized in the MN2 procedure. It is worth noting that the sign of the modal participating factor was retained in drift computation in the MN2 procedure to ensure that the drift components of the levels, where the higher modes control the responses, remain in the positive direction. It was shown that this assumption can improve the accuracy of the MN2 pushover analysis at the pile foundation part and the lower story levels of jacket offshore platforms (Mohamad Zarrin, Mostafa Gharabaghi, and Poursha 2020).

The accuracy of the proposed method in estimating the seismic demands of structures is validated by applying it to two sets of steel structures including three special moment frames (SMFs) with 6-, 12-, and 18-stories, and a 12-story SMF with soft stories (i.e. with stiffness irregularity over the height) as the first set, and two other steel buildings with 9-, and 20-story heights designed for the SAC Phase II Steel (Gupta and Krawinkler 1999) as the second set. Predictions resulting from the proposed method are compared with the results obtained by the NL-RHA as the benchmark solution as well as with those derived from the first-mode and the extended N2 pushover approaches. The evaluation is conducted in two seismic hazard levels.

Since performance-based earthquake engineering (PBEE) has become an interesting topic in the earthquake engineering sphere, several studies (Speicher and Harris 2016a, b; Balazadeh-minouei et al. 2017; Wang et al. 2017; Harris and Speicher 2018; Sattar 2018; Speicher and Harris 2018; Daei et al., 2021; Daei and Poursha, 2021) have been carried out in recent years to investigate the performance of different structures using the regulatory guidelines such as ASCE 41. However, the foregoing investigations were either based on the previous editions of ASCE 41 (ASCE 41-06 (American Society of Civil Engineers (ASCE) 2007) and ASCE 41-13 (American Society of Civil Engineers (ASCE) 2014)) or for performance assessment of RC buildings. Moreover, to the author's best knowledge, examining the accuracy and applicability of enhanced pushover procedures for seismic performance assessment of steel structures in the format of the ASCE 41 standard (American Society of Civil Engineers (ASCE) 2017)) has not been addressed yet. In this regard, the performance appraisal of the SMFs using the systematic evaluation framework of the new edition of ASCE 41 (ASCE 41-17) is set as the secondary objective of the current study.

2 Modal response analysis concept

The basic principles of the proposed procedure, which is based on the modal response history analysis, are outlined in the following section. The differential equation of motion governing the response of a nonlinear multi-degree-of-freedom (MDOF) system subjected to horizontal earthquake ground acceleration, $\ddot{u}_g(t)$, is as follows (Chopra 2001):

$$m\ddot{u} + c\dot{u} + ku = -mi\ddot{u}_g(t) \tag{1}$$

where m, c, and k are the mass, damping, and lateral stiffness matrices of the structure, respectively, and i is the unit vector. The right-hand side of Eq. (1) denotes the effective earthquake forces, $p_{\text{eff}}(t)$, and can be written as:

$$p_{\text{eff}}(t) = -mi\ddot{u}_g(t) = -s\ddot{u}_g(t) \tag{2}$$

where s is the spatial distribution of effective earthquake forces over the building's height. Eq. (2) can be obtained by expanding s as a summation of the modal inertia force distributions, s_n:

$$s = mi = \sum_{n=1}^{N} s_n = \sum_{n=1}^{N} \Gamma_n m \Phi_n \tag{3}$$

in which N is the number of the considered modes, Φ_n is the nth mode shape and Γ_n is the corresponding modal participating factor given by:

$$\Gamma_n = \frac{L_n}{M_n} = \frac{\Phi_n^T mi}{\Phi_n^T m \Phi_n} \tag{4}$$

The displacement vector of an N-degree-of-freedom system resulting from modal expansion can be defined as:

$$u(t) = \sum_{n=1}^{N} \Phi_n q_n(t) \tag{5}$$

where $q_n(t)$ is the modal coordinate and is determined by solving the dynamic equilibrium equation of the nth mode, so that:

$$\ddot{q}_n + 2\zeta_n \omega_n \dot{q}_n + \omega_n^2 q_n = -\Gamma_n \ddot{u}_g(t) \tag{6}$$

wherein ζ_n and ω_n denote the damping ratio of the system and the natural vibration frequency for the nth mode, respectively. The solution of Eq. 6 is as follows:

$$q_n(t) = \Gamma_n D_n(t) \tag{7}$$

where $D_n(t)$ is governed by the equation of motion for a single-degree-of-freedom (SDOF) system with vibration properties (i.e. natural frequency ω_n and damping ratio ζ_n) of the nth-mode of the MDOF system under $\ddot{u}_g(t)$:

$$\ddot{D}_n + 2\zeta_n \omega_n \dot{D}_n + \omega_n^2 D_n = -\ddot{u}_g(t) \tag{8}$$

Eventually, by substituting Eq. (7) into Eq. (5), the floor displacement vector of the system, $u(t)$, to the total excitation, $p_{\text{eff}}(t)$, can be formulated as follows:

$$u(t) = \sum_{n=1}^{N} \Gamma_n \Phi_n D_n(t) \tag{9}$$

The proposed method is established based on Eq. (9) to take into account the higher mode effects.

3 Description of the proposed procedure

The proposed procedure is a straightforward and less computationally demanding method that consists of two main steps. In the first step, a conventional pushover procedure is performed. Several methods including the basic N2 method (Fajfar 2000) implemented in Eurocode 8 (CEN. Eurocode 8 2004), a pushover procedure with a triangular or uniform load pattern according to FEMA 273 (FEMA 273 1997), or a pushover procedure with a lateral load distribution based on the first mode shape of the structure recommended by ASCE 41-17 (ASCE 41-17 2017) can be used for this purpose. It is well-established that all these methods are only able to yield reliable results for the low-rise buildings wherein the fundamental vibration mode dominates the seismic responses (Kreslin and Fajfar 2011). To fill this gap, a supplementary procedure should also be conducted. For this purpose, a second step based on structural dynamic theory is implemented in the proposed procedure to improve the shortcomings of traditional methods. Using this step, the contribution of higher modes to the response is incorporated. The key factor for this purpose is the methodology used for combining the interaction of several vibration modes. The quadratic modal combination rules such as SRSS or CQC are common ways to combine the peak modal responses of several modes to determine the total response quantity. However, using the quadratic modal combination rules neglects the effects of sign inversion due to the contribution of higher modes. Moreover, the quadratic modal combination rules, as is used in the MPA method (Chopra and Goel 2002), often overestimate the seismic demands (Kim and Kurama 2008). The absolute sum (ABSSUM) modal combination rule is another alternative to combine the maximum responses resulting from different modes. Similar to quadratic modal combination schemes, the ABSSUM method cannot retain the signs of the modal responses. Further, given the fact that the peak modal responses occur at different time instants, a direct combination of the modal responses of several modes disregarding their algebraic signs results in too conservative estimates. That is why the absolute sum combination method is not common for design purposes (Chopra 2001).

The proposed procedure employs another combination method in the second step to tackle the above-mentioned problems encountered when using the quadratic or absolute sum combination rules. In this method, the total structural responses are computed by the direct addition of the peak modal responses of the desired modes. Therefore, the employed combination rule is able to preserve the signs of structural responses for each mode, and accordingly, it can overcome the inherent conservatism associated with the absolute sum combination rule.

In general, in the NMP procedure, a conventional pushover analysis is first conducted, which governs the responses mostly at some floors in the lower part of the structure in which the fundamental vibration mode dominates the responses. After normalizing the desired seismic responses resulting from the pushover procedure to the top floor displacement, they are then modified by some correction factors to obtain the final results of the NMP procedure. These correction factors are obtained for story drifts through two equations based on the standard elastic modal response analysis. These expressions are virtually identical with the only difference that the signs of the modal participating factor (Γ_n) in computing modal story drifts are retained in one of the equations, while the mentioned signs are suppressed in the other. Since the direction of an earthquake is not definite and it can excite the structure in any direction, the adopted approach can somehow simulate the earthquake effects more realistically. After calculating correction factors for story drifts, they can be applied to other global or local seismic demands. It is noteworthy that the

correction factors mainly improve the results of the conventional pushover analysis at the upper part of the structure, where the higher mode effects are significant, while the results of some stories at the lowest part of some structures can also be modified using the computed correction factors.

The NMP procedure is composed of the following steps set forth below:

1. Conduct a conventional pushover analysis with an appropriate lateral load distribution (for example the basic N2 method (Fajfar 2000), the first-mode pushover analysis prescribed in ASCE 41 (American Society of Civil Engineers (ASCE) 2017), or a pushover analysis with a triangular or a uniform load pattern (American Society of Civil Engineers (ASCE) 2000)) until the displacement of the control node at the top floor level reaches an appropriate predefined target roof displacement. Determine the peak values of the desired responses, namely story displacements, story drifts, plastic hinge rotations, etc., for the pushover analysis at the target displacement.

2. Conduct the standard elastic modal response analysis for all the modes of interest. For this purpose, the following sub-steps should be carried out:
2.1 Calculate the natural frequencies, ω_n, mode shapes, $\mathbf{\Phi}_n$, and modal participation factors, Γ_n, for the considered vibration modes through eigenvalue analysis of the linearly elastic system. The mode shapes are normalized such that the roof component of $\mathbf{\Phi}_n$ equals unity.
2.2 Compute the jth story displacement, U_j, by combining the modal story displacements, u_{jn}, through algebraic sum including the interaction of k vibration modes:

$$U_j = \sum_{n=1}^{k} u_{jn} \ with \ u_{jn} = |\Gamma_n| \mathbf{\Phi}_{jn} S_{dn}(\zeta_n, T_n) \tag{10}$$

where S_{dn} is the elastic spectral displacement as a function of vibration period, T_n, and damping ratio, ζ_n, of the nth mode for a given earthquake ground motion.
2.3 Compute the story drift ratio of the jth story, Δ_{jn}, using both of the following equations:

$$\Delta_j = \sum_{n=1}^{k} \left(\frac{u_{jn} - u_{j-1,n}}{h_j} \right) \tag{11}$$

$$\overline{\Delta}_j = \sum_{n=1}^{k} \left(\frac{\overline{u}_{jn} - \overline{u}_{j-1,n}}{h_j} \right) \ with \ \overline{u}_{jn} = \Gamma_n \mathbf{\Phi}_{jn} S_{dn}(\zeta_n, T_n) \tag{12}$$

where h_j is the height of the jth story.
2.4 Normalize the results obtained in sub-steps 2.2 and 2.3 such that the roof displacement of structure (calculated using Eq. (10)) equals the target displacement used for the pushover analysis in the 1st step.

3. Calculate the final peak responses of the structure by enveloping the results obtained in the 1st and 2nd steps as the following sub-steps:
3.1 Determine the correction factors for each story level via dividing the normalized story drift ratios resulting from the elastic modal response analysis (the 2nd step) by those

obtained by the pushover analysis (the 1st step). Note that the maximum values of the correction factors obtained using each of the Eqs. (11) and (12) are considered for the subsequent calculations. The ultimate correction factors must be greater than or equal to 1.0.

3.2 Calculate peak seismic demands by multiplying the results obtained in the 1st step by the corresponding correction factors calculated in the previous sub-step 3.1.

4 Validation of the proposed procedure

The validation of the accuracy and effectiveness of the NMP procedure in predicting the seismic demands of structures is carried out by examining it with several steel moment frames with different heights. This verification is carried out by comparing the results of the proposed procedure with the relevant results corresponding to the nonlinear response history analysis (NL-RHA) as the most accurate analysis method. Further, for the sake of comparison, the predicted results obtained by a conventional pushover method with a lateral force distribution based on the first mode shape of the structure (Mode-1), and the extended N2 method are also presented.

4.1 Illustrative examples

In the current work, two sets of representative steel structures are adopted. The first set contains four buildings with different heights that are designed according to the provisions of ASCE 7-16 (ASCE 7-16, 2016), AISC 360-16 (AISC 360-16 2016a, b), and AISC 341-16 (AISC 341-16, 2016a, b). Three of the four buildings in the first set are regular frames having 6, 12, and 18 stories. The last building of the first set is a structure with somehow stiffness irregularity along the height, which is created by modifying the member sizes of the 12-story building in this suite. This new building, which is labeled 12-story-SS, is generated by decreasing the beam member sizes at the 9 and 10 stories and increasing them at the other stories. To satisfy the prescriptive requirements of the previously mentioned standards, the column member sizes are also changed from W18 in the reference frame to W24.

All the buildings in the first set have three bays with 6 m width in both directions. The base story height is 4 m and all other stories have a typical story height of 3.5 m. Building stability and resistance to lateral loads and deformations are provided by external special moment frames (SMFs) along the north-south (N-S) direction and special concentrically braced frames (SCBFs) along the east-west (E-W) direction on the perimeter of the building. The interior frames only resist the gravity loads. The focus of this paper is only on SMFs. The buildings are located in Sunnyvale, California (latitude/longitude = 37.404/-122.002), which is considered a high-seismicity region. It is assumed that the archetypes are constructed on stiff soil (site class D). All beams and columns are made of A992 (American Society for Testing and Materials (ASTM) 2015) steel, with nominal yield stress F_y of 345 MPa and tensile strength of F_u of 450 MPa. Wide-flange sections are used for all members. Panel zones are considered in the models and doubler plates are added to columns' web within the joint region according to the provisions of AISC 341-16 (AISC 341-16, 2016a, b). The floor dead load is 4.032 kN/m^2 and 3.71 kN/m^2 for the typical stories and roof, respectively. A 9.61 kN/m superimposed dead load is also applied to the perimeter beams to consider the weight of perimeter walls. This load is 1.44 kN/m for

the roof. The interior walls (i.e. partitions) dead load is taken 1.72 kN/m². The live load of the floors and the roof is 1.96 kN/m² and 1.47 kN/m², respectively. Snow load is 0.49 kN/m². The loading width of the frames is 9 m. Seismic mass is computed as the dead loads plus 20% of the live load and 20% of the snow load. The configuration of the frames in the first set and their column and beam section dimensions are demonstrated in Fig. 1. In this figure, the thickness of doubler plates and the assembled joint mass are also provided. Because of the symmetry of the frames, the mentioned properties are only presented for half of the structures.

The second set of structures used to reinforce the investigation contains the Seattle 9-, and 20-story SAC buildings (Gupta and Krawinkler 1999; Ohtori et al. 2004), denoted by SAC-9 and SAC-20, respectively. These structures prepared by three consulting firms as a part of the SAC Phase II Steel Project (FEMA 355C, 2000) have been widely used by researchers. The buildings were office buildings located on stiff soil. The moment-resisting frames (MRFs) in the north-south (N-S) direction of these buildings are considered for this study. A572 Gr. 50 steel with a nominal yield strength of 345 MPa and expected yield strength of 397 MPa were used for both columns and beams. Further details of SAC steel buildings can be found in (Gupta and Krawinkler 1999; Ohtori et al. 2004).

4.2 Numerical modeling

In this study, the simulation of archetype frames is performed by OpenSees (OpenSees 2016). The lumped plasticity approach is utilized to model the mechanical behavior of the beam and column elements considering the strength and stiffness deterioration of the members under cyclic loading using the modified Ibarra-Medina-Krawinkler (IMK) deterioration model developed by Lignos and Krawinkler (Lignos and Krawinkler 2012). This deterioration model is a customized version of the IMK deterioration model (Ibarra and Krawinkler 2005). The fundamental characteristics of this model are a backbone curve, a set of rules establishing the hysteretic behavior, and several rules defining the deterioration rate. The backbone curve of this model is presented in Fig. 2, wherein M_y and θ_y are the effective yield strength and rotation, respectively, K_e is the effective elastic stiffness, M_c and θ_c are the capping strength and rotation, respectively, θ_p is the pre-capping rotation, θ_{pc} is the post-capping rotation, M_r is the residual strength and θ_u is the ultimate rotation where the tearing of the section is initiated.

Lignos and Krawinkler (Lignos and Krawinkler 2011) utilized the results of a group of experiments performed on wide-flange sections as beam sections to quantify the important parameters that affect the cyclic moment-rotation relationship at plastic hinge regions in beams. Using the information derived from this steel component database, empirical relationships for modeling the backbone parameters of the modified IMK deterioration model are developed in their study, which are utilized herein for the accurate prediction of the hinge hysteretic behavior. Moreover, the new modeling criteria for the first-cycle envelope and monotonic backbone curves of steel wide-flange columns for use in nonlinear static and dynamic analyses, respectively, proposed by Lignos et al. (Lignos et al. 2019) recently are utilized for column hinge properties.

The frames are modeled with elastic beam-column elements connected by zero-length elements, which serve as rotational springs to represent the hinges' nonlinear behavior using the modified IMK Model. For the first set of structures, panel zones are explicitly modeled using the approach proposed by Gupta and Krawinkler (A. Gupta and Krawinkler 1999) as a rectangle composed of eight very stiff elastic beam-column elements and one
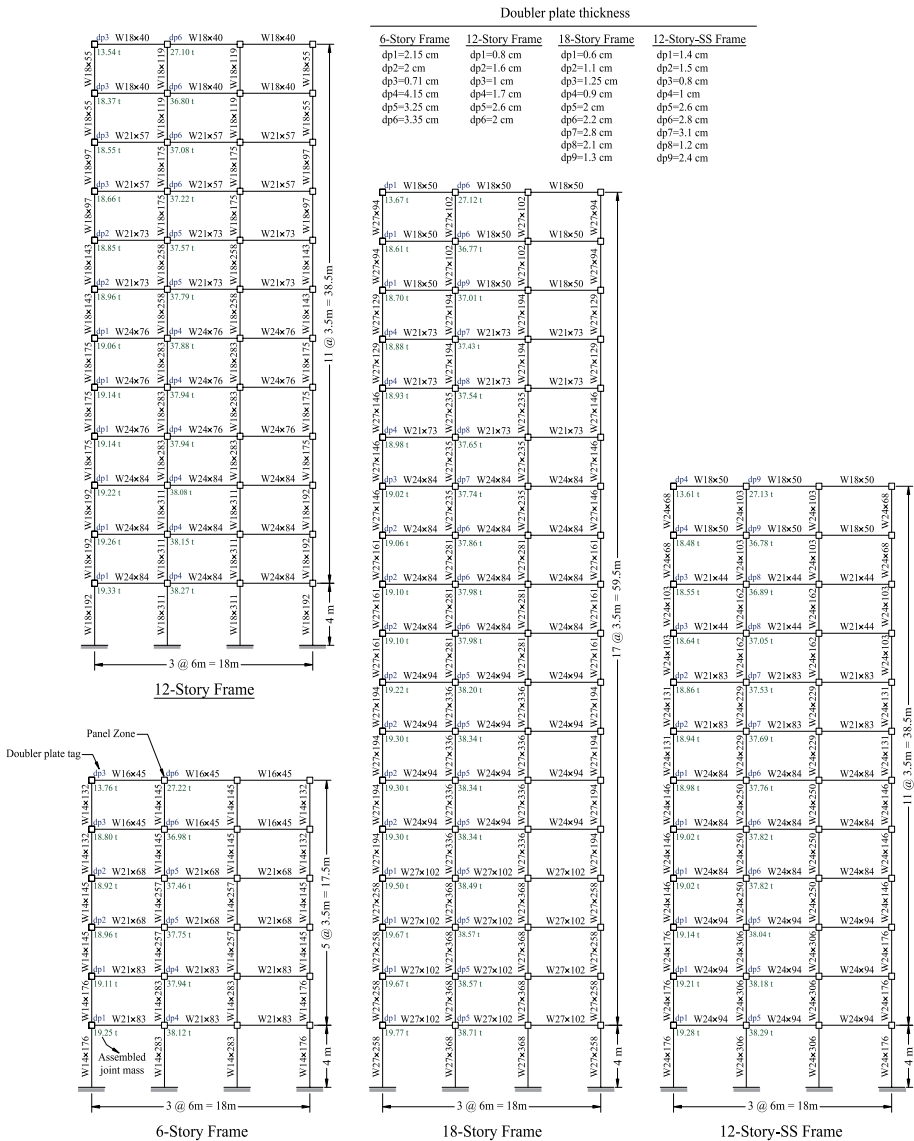
Fig. 1 Details of the first set of structures used in this study

zero-length element, which serves as a rotational spring to represent shear distortions in the panel zone. The schematic representation of the numerical modeling is illustrated in Fig. 3. In the case of SAC buildings, a basic centerline modeling of the bare MRF (denoted by "Model M1" utilized by Gupta and Krawinkler (Gupta and Krawinkler 1999)) is adopted. Therefore, panel zones are excluded from the modeling of these structures. The natural vibration periods of the frames obtained by OpenSees along with the modal mass participation ratios (α_n) and modal participating factor (Γ_n) for the first three modes are provided in Table 1.

Fig. 2 Modified Ibarra-Krawinkler (MIK) deterioration model backbone curve
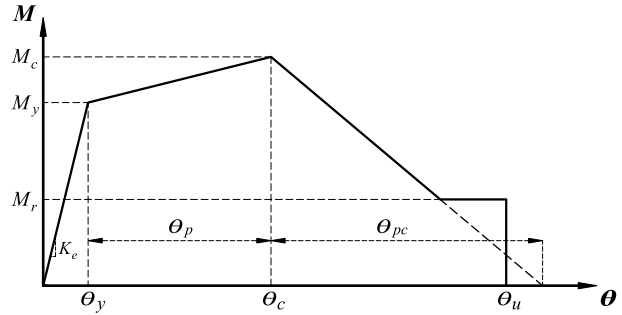
4.3 Ground motions for nonlinear response history analysis

For the first set of buildings, a structure-specific and site-specific ground motion record selection procedure is conducted independently for each of the buildings for performing the NL-RHA. The target spectrum used in this paper is a Uniform Hazard Spectrum (UHS) constructed from hazard curves resulting from Probabilistic Seismic Hazard Analyses (PSHA). The 2% in 50 years UHS hazard curve has been derived using the unified hazard tool of the United States Geological Survey (USGS) (https://earthquake.usgs.gov/hazards/interactive). According to chapter 21 of ASCE 7-16 (ASCE 7-16 2016), the risk-targeted maximum considered earthquake (MCE_R) spectrum is determined by multiplying the UHS hazard curve by a risk coefficient, C_R. The design response spectrum is also computed by multiplying the MCE response spectrum by 2/3 factor. Different records are selected for the design basis earthquake (DBE) and MCE levels. For each structure at each hazard level, 20 far-fault ground motion records are selected from the strong ground motion database (NGA-West2) of the Pacific Earthquake Engineering Research (PEER) center (https://ngawest2.berkeley.edu) using the
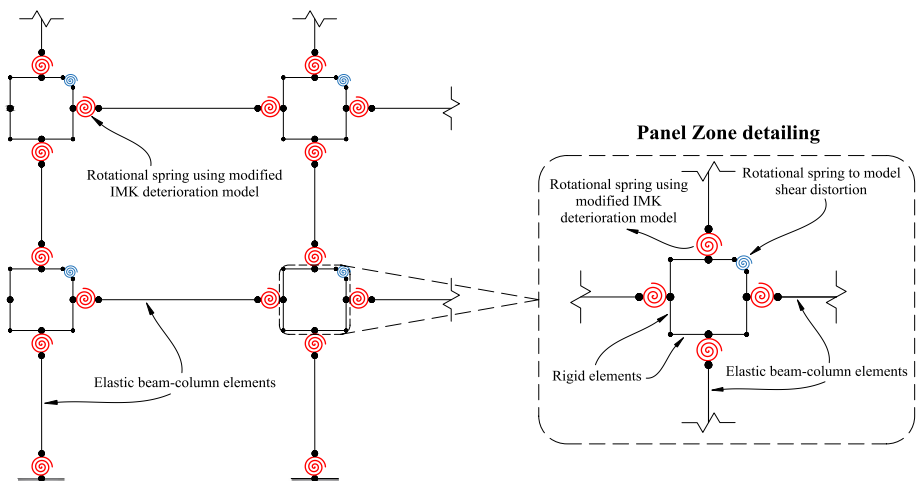

Fig. 3 Schematic representation of nonlinear modeling strategy employed in the first set of buildings including explicit modeling of the panel zones. (Note: the same modeling approach without panel zones used for the second set of buildings)

Table 1 Modal properties of the example buildings

Frame	Periods			Modal participating mass ratios (α_n)			Modal participating factor (Γ_n)		
	T_1(s)	T_2(s)	T_3(s)	α_1	α_2	α_3	Γ_1	Γ_2	Γ_3
6-Story	1.030	0.371	0.195	0.795	0.123	0.049	1.401	−0.588	0.275
12-Story	1.795	0.656	0.372	0.763	0.123	0.046	1.417	−0.652	0.389
18-Story	2.258	0.796	0.453	0.726	0.135	0.052	1.442	−0.699	0.431
12-Story-SS	1.640	0.609	0.328	0.741	0.136	0.052	1.443	−0.661	0.371
SAC-9	3.124	1.119	0.604	0.810	0.127	0.036	1.362	−0.526	0.247
SAC-20	3.886	1.393	0.809	0.769	0.139	0.043	1.435	−0.657	0.340

single-period (the fundamental period of the structure, T_1) scaling method. This number of selected ground motions for each building is more than the minimum 11 records prescribed by ASCE 7-16. The selection procedure is done according to the provisions of the ASCE 7-16. In this regard, the records are selected such that no individual record has a scale factor lower than 0.25 and greater than 4. Further, no more than four ground motion records are selected from a single earthquake event. All the selected ground motions are from events with a moment magnitude, M_w, between 6.4 and 7.62, representing mid to large magnitude earthquakes. The distance from the fault rupture for the selected records is between 15 and 32 km for the DBE level and 11 and 49 km for the MCE level. Note that the records with a distance between 10 and 15 km must have a magnitude lower than 7.0 according to ASCE 7-16 to be considered as far-fault. All the records were recorded on stiff soil (site class D) with average shear wave velocity (V_{s30}) of 183 m/s to 365 m/s.

In the current work, the ground motion intensity is scaled by the spectral acceleration at the fundamental period of the structure, $S_a(T_1)$. Figure 4 illustrates the elastic spectra of the selected record ensembles along with the target DBE or MCE_R spectrum. As is evident, a good match is observed between the geometric mean (Abyani et al. 2019; Zarrin *et al.* 2020a, b) spectra of the selected records and the target DBE or MCE spectrum in the period range of $0.2T_1$ to $2T_1$ recommended by ASCE 7-16. It is noteworthy that the average spectrum of the records does not fall below 90% of the target response spectrum for any period within the prescribed period range (refer to chapter 16 of ASCE 7-16). The list of selected records for the buildings in the first set of structures at the DBE and MCE levels is tabulated in Table2. Only the record sequence number (RSN) and the ground motion component are provided in this table.

In the case of the second set of structures (i.e. SAC buildings), the original records assembled for the SAC Phase II project (Gupta and Krawinkler 1999) for a site with a stiff soil profile in Seattle are used. These ground motions are recorded and simulated far-fault records (SE01-SE40) in two hazard levels having probabilities of exceedance of 10% (corresponds to the DBE level) and 2% (corresponds to the MCE level) in 50 years. Each hazard level contains 20 records i.e. 10 ground motions each of which with two orthogonal components. These ground motions were scaled such that the average of the spectra of the two horizontal components matches the given target spectrum with the weighted minimum square error at the 0.3 s, 1 s, 2 s, and 4 s periods. For more detailed information about the ground motions, refer to (Gupta and Krawinkler 1999).
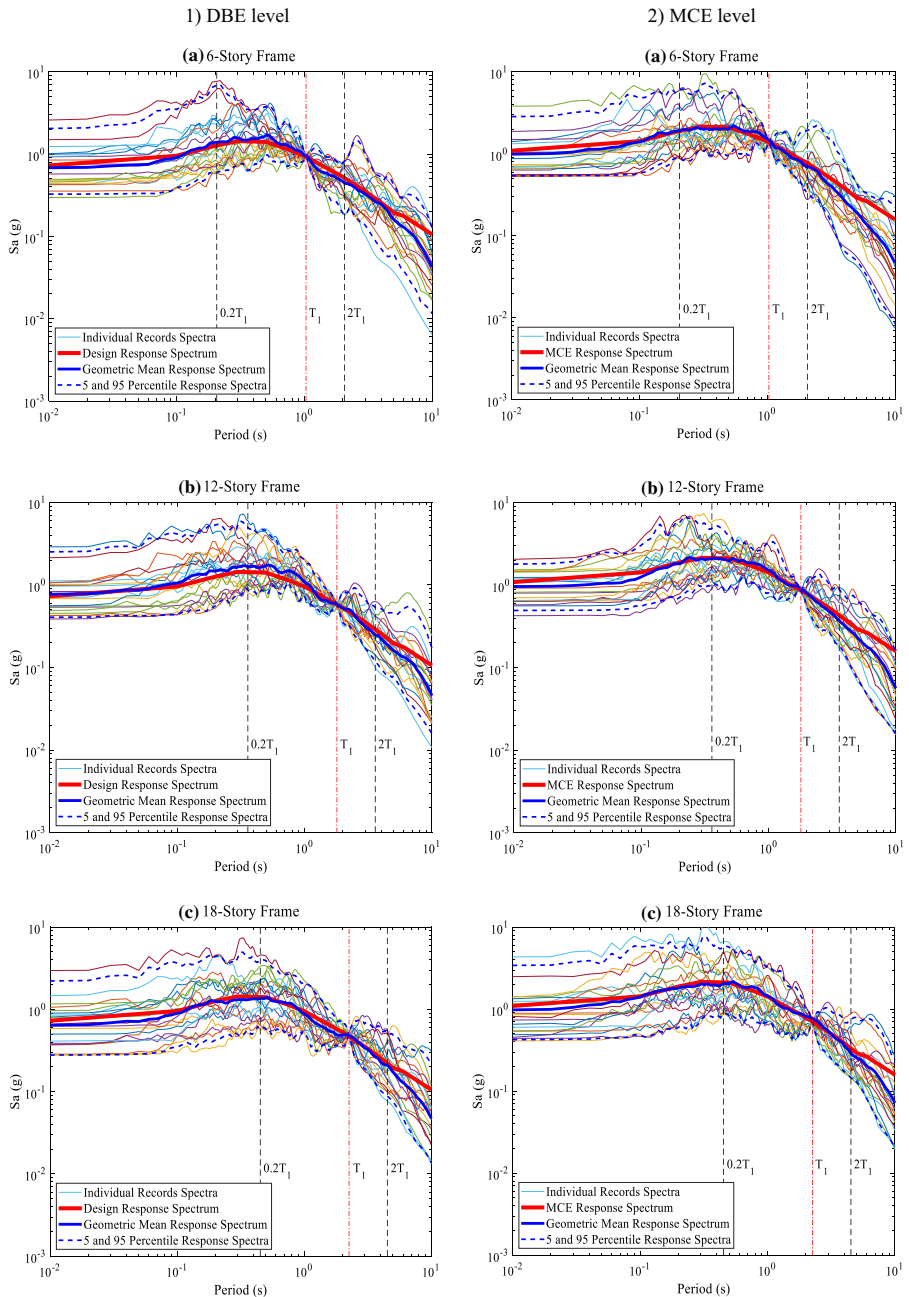
1) DBE level   2) MCE level

Fig. 4 The pseudo-acceleration response spectrum for individual records along with the target response spectrum (DBE or MCE), the geometric mean response spectrum of selected records, and the 5 and 95 percentile response spectra for the first set of structures

Table 2 List of the ground motions selected for the first set of structures at the DBE and MCE levels

No.	DBE level						MCE level					
	6-story		12-story		18-story		6-story		12-story		18-story	
	RSN	Comp	RSN	Comp	RSN	Comp	RSN	Comp	RSN	Comp	RSN	Comp
1	721	0	169	262	169	262	752	0	174	140	174	140
2	776	0	721	0	721	0	776	0	338	0	776	0
3	777	90	776	0	778	165	777	90	776	0	778	165
4	778	165	777	90	786	55	953	9	777	90	806	270
5	786	55	778	165	953	9	1087	90	778	165	953	9
6	953	9	953	9	1077	90	1158	180	960	0	1087	90
7	1077	90	1077	90	1087	90	1187	N	1104	0	1101	0
8	1158	180	1087	90	1158	180	1203	E	1158	180	1180	N
9	1194	E	1104	0	1194	E	3754	90	1194	E	1246	N
10	1203	E	1110	0	1238	N	4894	NS	1203	E	1537	E
11	3749	270	1158	180	1543	E	4895	NS	1542	E	1547	E
12	5814	NS	1180	N	3749	270	4896	NS	4207	NS	3749	270
13	5837	360	3749	270	4861	NS	5664	NS	4894	NS	4861	NS
14	5985	360	5827	0	5814	NS	5786	NS	4895	NS	4894	NS
15	5991	320	5832	0	5823	0	5823	0	5786	NS	5774	NS
16	5992	360	5837	360	5832	0	5827	0	5827	0	5827	0
17	6888	N26W	5992	360	6889	N01W	5832	0	5992	360	5829	0
18	6889	N01W	6888	N26W	6890	N10E	5991	320	6888	N26W	5831	0
19	6890	N10E	6890	N10E	6923	N15E	6888	N26W	6923	N15E	5832	0
20	6923	N15E	6923	N15E	6952	S33W	6890	N10E	6952	S33W	6952	S33W

4.4 Details of the analyses

The reference seismic demands of the example frames are determined as the mean values of the maximum seismic demands resulting from the NL-RHAs with the aid of the selected ground motion records. A step-by-step numerical integration based on Newmark's average acceleration method is used for conducting NL-RHAs. The stiffness and mass proportional damping are assigned to the elastic beam and column elements and the nodes containing mass, respectively. A Rayleigh damping ratio of 2% is enforced at the first and third vibration modes for the first set of structures and the SAC-9 building, and the first and fifth modes for the SAC-20 building. As mentioned above, besides the proposed NMP method, two other pushover procedures including a conventional pushover procedure using the first-mode pushover procedure prescribed in ASCE 41 (ASCE 41-17 2017) and the extended N2 method are also performed for comparison purposes. It is noteworthy that the first step of the extended N2 and the NMP procedures are conducted using the load pattern based on the first-mode shape vector. Note that the nonlinear analyses are carried out after applying gravity loads. The target roof displacement for all the pushover procedures is set equal to the mean value of the peak absolute floor displacements resulting from the NL-RHAs for the selected ground motion ensembles at each hazard level. This assumption helps to minimize the further approximation in comparing the results of different pushover methods (Kreslin and Fajfar 2011, 2012).

5 Discussion of the results

The following section presents the results of the numerical study by thoroughly evaluating the performance of the proposed pushover procedure with respect to the estimates obtained by the NL-RHA, as the benchmark solution, and two other approximate methods. The interstory drift ratio (IDR) and the plastic hinge rotation of beams at each floor level are two important engineering demand parameters (EDPs) used in the current study to evaluate the accuracy of the proposed pushover procedure. A performance evaluation is also carried out based on the systematic evaluation procedure of ASCE 41-17 (ASCE 41-17 2017) to appraise the efficiency of the proposed method in seismic performance assessment of the structures. The results of each EDP are provided in different subsections for both the DBE and MCE hazard levels. Before presenting the validation results, the pushover curves associated with the first mode of vibration for all the example buildings are provided and discussed.

5.1 Pushover curves

Figure 5 shows the pushover curves of the test buildings pushed by a lateral load proportional to the first mode of vibration until a large displacement. In this figure, the target roof displacements of the studied buildings at the DBE and MCE levels are also illustrated. This kind of representation is greatly helpful to figure out to what extent the structures step into the inelastic range during the pushover analysis. The pushover curves indicate that the SAC-9 frame at the DBE level and the SAC-20 frame at both the DBE and MCE levels almost remain elastic. It is evident that the maximum roof displacement of the SAC-20 frame at the DBE level is far below the yield displacement. This behavior demonstrates that no or negligible plastic mechanism is formed in the SAC-20 building, which may lead to significant errors in the estimations of the approximate methods. Conversely, the buildings in the first set at both hazard levels and the SAC-9 frame at the MCE level deform well beyond the elastic limit.

5.2 Interstory drift ratios

It was previously explicated in Sect. 3 that the NMP procedure utilizes a first-mode-based pushover analysis and two equations (Eqs. (11) and (12)), based on the standard elastic modal response analysis, for determining the interstory drift ratios. Both the pushover analysis and the proposed equations are of the essence to obtain acceptable structural responses all over the structure's height because the envelope of the results is taken as the final seismic demands. To further probe into the significance of the first-mode pushover analysis and each of Eqs. (11) and (12) (normalized in step 2.4 of Sect. 3) in the proposed procedure, the interstory drift ratios obtained from them along with those from NL-RHA for the 12-story frame are shown in Fig. 6. As can be seen in this figure, the NMP procedure using Eq. (12) and the Mode-1 pushover analysis controls the result at the lower and some intermediate stories, whereas the IDRs obtained by Eq. (11) are closer to the results of the NL-RHA at the upper part of the investigated building. To be precise, Eq. (12) at the first two lower stories, the Mode-1 pushover analysis from the 3rd up to the 7th stories, and Eq. (11) at the top five floor levels control the response. This reflects that the Eq. (12) helps to improve the results of pushover analysis at some stories located at the lowest part of the structure, while Eq. (11) enhances the IDRs at upper story levels, where the effects of
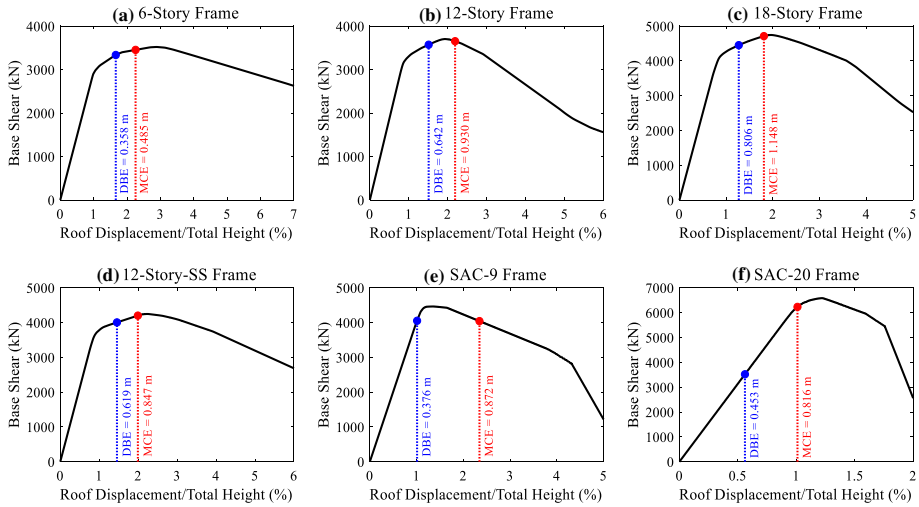
Fig. 5 The first-mode-based pushover for the six example buildings; the target displacements at the DBE and MCE levels are also demonstrated in each plot

higher modes are notable. The same interpretation can also be put on the plastic hinge predictions, which will be discussed in the next section. It is noteworthy that similar findings, as described for the 12-story frame, are observed for the other frame models considered in this study.

The interstory drift ratios of the example buildings resulting from the Mode-1, extended N2, and NMP pushover procedures along with the NL-RHA results are depicted in Figs. 7 and 8 for the DBE and MCE levels, respectively. As expected, the conventional Mode-1 pushover analysis lacks the ability to accurately estimate the IDRs, by grossly
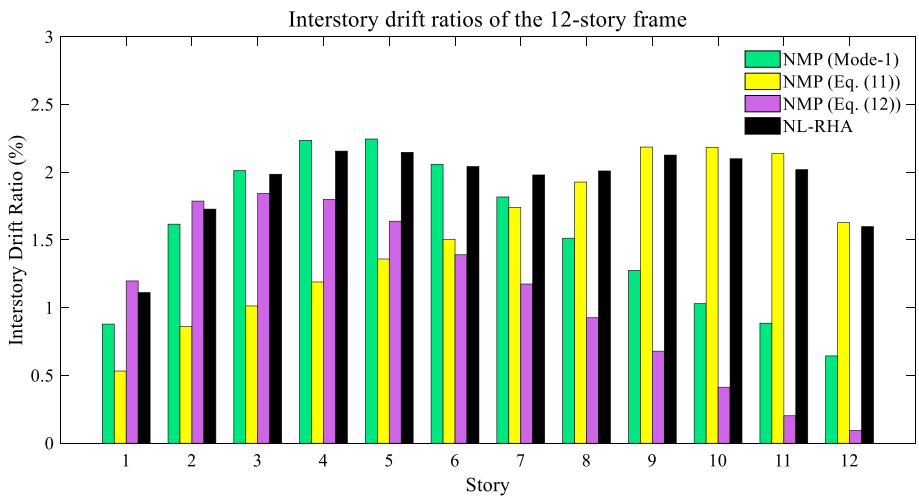
Fig. 6 The interstory drift ratios of the 12-story frame obtained by the Mode-1 pushover analysis and Eqs. (11) and (12) used in the NMP procedure along with those obtained by the NL-RHA at the DBE level

underestimating them at the upper stories. The single-mode pushover method cannot even predict the demands with acceptable accuracy in the case of the 6-story frame, wherein the higher mode effects are not highly significant. This holds true for both seismic hazard levels. On the other hand, the proposed NMP method achieves a near-perfect prediction of the IDRs almost at all stories for all the frames at the DBE level. As such, the story drift profile of the NMP method perfectly matches the results of the NL-RHA. The extended N2 method tends to underestimate the story drifts at the upper part of the test buildings at the DBE level, however, the overall estimates of this method are almost satisfactory. As can be seen in the figures, the extended N2 and NMP procedures provide somewhat identical estimates of the IDRs, with very few exceptions, at the lower and intermediate floor levels in which the first vibration mode dominates the responses. This is because of the quite similar concept of these two methods in estimating the demands at the lower parts of the structure.

The results demonstrate that the accuracy of the enhanced pushover procedures in estimating the IDRs varies by increasing the earthquake intensity. Figure 8 illustrates that, at a higher earthquake intensity (the MCE level), the accuracy of the NMP procedure in estimating story drifts is still highly satisfactory in most cases. However, the
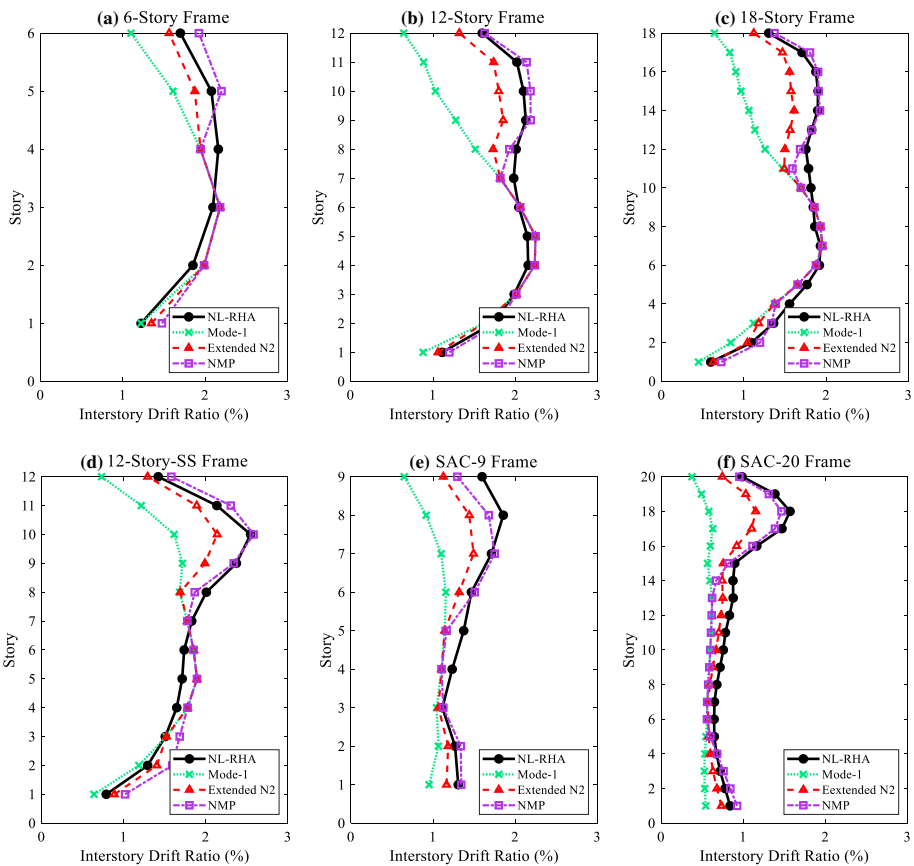

Fig. 7 The interstory drift ratios of the frames resulting from the NMP, extended N2, and Mode-1 pushover procedures as well as the NL-RHA at the DBE level
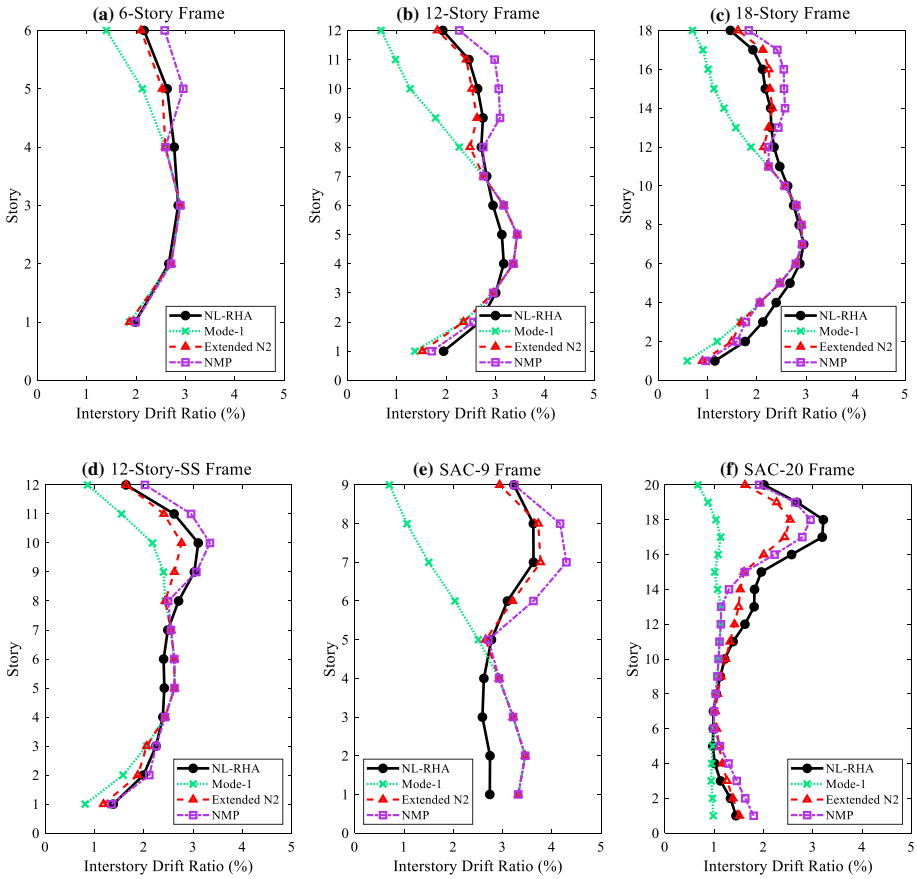
Fig. 8 The interstory drift ratios of the frames resulting from the NMP, extended N2, and Mode-1 pushover procedures as well as the NL-RHA at the MCE level

IDRs derived using the NMP procedure are moderately conservative at the upper floor levels for all the archetypes, except the SAC-20 building. Nevertheless, the differences between the NMP method and the benchmark solution (NL-RHA) are not significant and the results are acceptable. Except for the 12-story-SS and SAC-20 frames, the best estimates of the IDRs at the MCE level are provided by the extended N2 method, especially at the upper stories. In the case of the 12-story-SS frame, the results obtained by the NMP method at the first three lower stories and some upper floors are closer to the benchmark results. Similarly, in the SAC-20 building, the NMP procedure gives relatively better estimates of IDRs at the upper part of the structure compared to the extended N2 method, while the accuracy of the extended N2 method at some lower and intermediate stories is marginally better than the proposed procedure. Generally speaking, despite the discrepancies between the results of the enhanced pushover methods evaluated in this study (i.e. the extended N2 and NPM) and the NL-RHA, they can appropriately take into account the higher mode effects in the prediction of the story drifts at the upper stories of the buildings wherein these effects are prominent.

A more detailed inspection of the results for the 12-story-SS, SAC-9, and SAC-20 frames with a non-uniform distribution of the IDRs reveals that the NMP estimates for such stiffness-irregular frames are almost as accurate as those obtained for the other frames at both intensity levels. This indicates that the presence of a soft story in these frames does not degrade the accuracy of the proposed procedure. Moreover, the NMP procedure can properly identify the location of the peak story drift value over the building's height. Strictly speaking, the proposed procedure is consistently capable of reproducing the accurate height-wise distribution of seismic demands obtained by the most accurate method, the NL-RHA.

5.3 Plastic hinge rotations

The height-wise variation of plastic hinge rotations of the beams is illustrated in Figs. 9 and 10 for the DBE and MCE levels, respectively. These values are expressed as the maximum value of the plastic rotations for all beam hinges at each story level. The plastic rotation estimates obtained by the Mode-1 analysis are relatively acceptable at the lower and some intermediate stories, especially for the first set of structures. However, this method yields extremely flawed predictions at the upper part of the structures. While both the extended N2 and NMP procedures utilize the same methodology (first-mode pushover analysis) to estimate the demands at the lower part of the structure, the results of these procedures at some lower stories are different. As such, the estimates of the NMP procedure at some lower floors of the buildings are systematically greater than those of the extended N2 method at both the intensity levels. This finding holds also true for the interstory drift ratios presented in the previous subsection. This is due to the larger correction factors (refer to Sect. 3) obtained for the NMP method.

compared to the extended N2 method at the first three or four lower levels. For these floor levels, except for the SAC-20 frame, the extended N2 procedure provides a relatively better estimation of the plastic rotation demands than its conceptual counterpart at the DBE level. Conversely, it is the NMP method that is in closer agreement with the benchmark solution in most cases at the MCE intensity level.

When moving to the upper floors, the differences between the results of different pushover methods significantly increase. By inspecting Figs. 9 and 10, it can be perceived that the NMP procedure has appreciable superiority over the two other approximate methods in estimating the plastic hinge rotations of the elements at the upper part of the case studies. In other words, the results of the NMP procedure are in closer agreement with the NL-RHA compared to the other approximate NSPs for the entire archetypes. However, at the DBE level, the plastic rotations at the upper stories and some intermediate levels are generally underestimated by the proposed method. Except for the SAC-20 building, an increase in the earthquake level (i.e. the MCE level) leads to a perceptible improvement in the accuracy of the NMP method in estimating the local demands at the upper part of the structure. This is in contrast to the IDR demands (see Figs. 7 and 8) in which the NMP procedure has relatively better performance at the DBE level than the MCE level. The larger differences between the NMP method and the NL-RHA in the SAC-20 frame in comparison to the other buildings are because this building remains almost elastic, even at the MCE level (see Fig. 5 f). This may be attributed to the fact that the SAC-20 frame is mainly designed to resist wind loading (A. Gupta and Krawinkler 1999). Nonetheless, the correction factors used in the NMP procedure significantly improve the results in comparison with the other
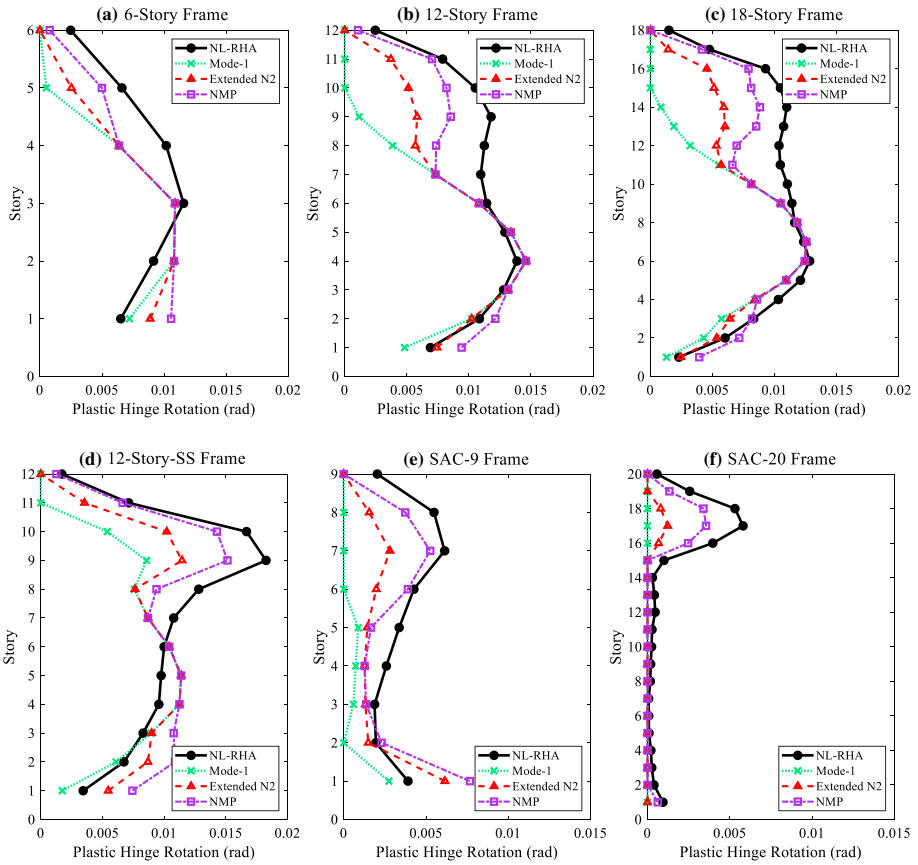
Fig. 9 The plastic hinge rotations of the beams resulting from the NMP, extended N2, and Mode-1 pushover procedures as well as the NL-RHA at the DBE level

methods. It is also worth mentioning that the plastic rotation estimates obtained by the extended N2 method are more satisfactory for the higher earthquake intensity.

On the whole, the NMP procedure provides more accurate estimates of the plastic rotations than the extended N2 procedure for all the archetypes at both earthquake intensity levels, all things considered. This indicates that the algebraic sum of the modal responses used in the NMP procedure results in better estimation of local demands compared to the quadratic modal combination rules employed by the extended N2 method. This is mainly due to the fact that the sign of each modal response is preserved and the sign reversal is included in the ultimate response of the structure when using the NMP method.

5.4 Performance evaluation using ASCE 41

Performance-based earthquake engineering (PBEE) has been evolving as a desirable philosophy for the design of new buildings and the seismic evaluation and upgrade of existing structures (either an existing building or a completed design of a new building). PBEE is a generalized methodology in which the seismic design and evaluation of
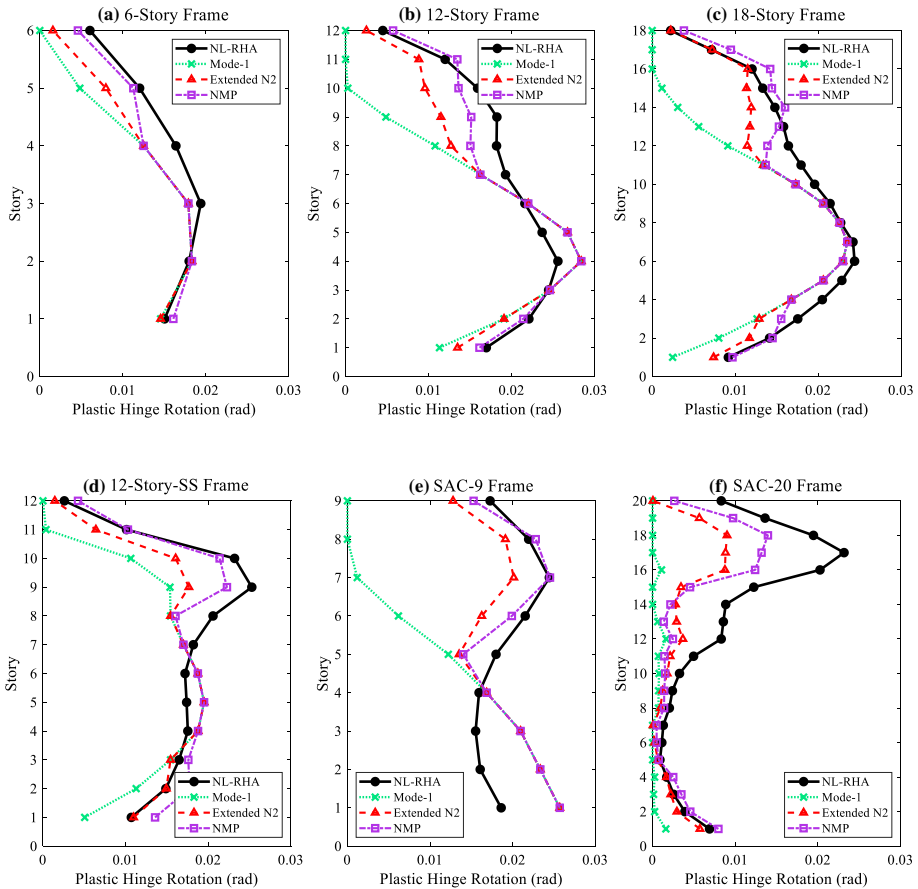
Fig. 10 The plastic hinge rotations of the beams resulting from the NMP, extended N2, and Mode-1 pushover procedures as well as the NL-RHA at the MCE level

buildings are accomplished based on multiple performance objectives (Ghobarah 2001; Krawinkler and Miranda 2004). This systematic methodology provides a framework to quantitatively predict the responses of structures, levels of safety, and economic losses in case of possible risks that cannot be fulfilled using traditional prescriptive approaches (Deierlein et al. 2003; Krawinkler and Miranda 2004). ASCE 41 (ASCE 41-17 2017) is a valuable reference for implementing the first-generation PBEE procedure into the seismic evaluation and design of buildings. This standard provides a systematic evaluation methodology in which the performance assessment is performed using four different analysis procedures including linear static procedure (LSP), linear dynamic procedure (LDP), nonlinear static procedure (NSP), and nonlinear dynamic procedure (NDP). ASCE 41 has been widely used by practitioners and researchers. In recent years, several efforts have been conducted to evaluate the performance of different varieties of buildings using the ASCE 41 standard (Speicher and Harris 2016a, b; Balazadeh-minouei et al. 2017; Wang et al. 2017; Harris and Speicher 2018; Sattar 2018; Speicher and Harris 2018; Daei et al., 2021; Daei and Poursha 2021).

In this section, the performance of the example buildings is evaluated using the latest edition of ASCE 41, i.e. ASCE 41-17. This evaluation is conducted only for the first set of structures designed based on the recent seismic codes. The main objective of this section is to investigate the applicability and validity of the two enhanced pushover procedures used in this study in evaluating the performance of the buildings and comparing their results with the NSP and the NDP procedures prescribed by ASCE 41. The ASCE 41 evaluation process generally starts by selecting the performance objective. In this study, the basic performance objective equivalent to new building standards (BPON) is used as the target performance objective. This target performance objective has the dual goal of life safety (LS) at the basic safety earthquake-1 N (BSE- 1 N) and collapse prevention (CP) at the basic safety earthquake-2 N (BSE-2 N) for structures with Risk Category I or II. The ground motion records scaled to MCE_R and DBE levels are considered directly as the BSE-2 N and BSE-1 N seismic hazard levels, respectively.

As mentioned earlier, the accuracy of the two enhanced NSPs and the NSP prescribed by ASCE 41 in estimating the performance of the structures is evaluated with the NDP procedure as the benchmark solution. The pushover procedure recommended by ASCE 41 or the other guidelines only takes into account the first mode effect in response, and the contribution of higher modes is neglected. However, ASCE 41 necessitates a supplementary analysis procedure in the presence of higher mode effects. That is, for the structures with substantial higher mode effects, the use of first-mode-based NSP is permitted by ASCE 41 only if an LDP analysis is also executed to supplement the NSP. Buildings with significant higher mode effects must meet the acceptance criteria of ASCE 41 for both the NSP and LDP (ASCE 41-17, 2017). The higher mode effects are considered significant if the ratio of the shear at any story, obtained by the modal response spectrum analysis considering all modes with at least 90% modal mass participation ratio, to the corresponding story shear computed by the analysis considering only the fundamental mode of the structure exceeds 1.3. This provision is checked for the archetype frames, and the mentioned story shear ratios are plotted in Fig. 11. The dashed lines in this figure demonstrate the 130% limiting value proposed by the standard. It can be seen in the figure that the shear ratio for all the frames, except for the 6-story frame, is greater than the 1.3 limiting value at some upper stories. Therefore, the higher mode effects in these frames are significant, and the supplementary LDP evaluation must be conducted for them accordingly. In the current work, the LDP analysis is carried out using the response history method.

In order to evaluate the seismic performance, the normalized demand-to-capacity ratios (DCR_N) of flexural actions in the beam and column components are computed. For a given performance level, the DCR_N values are the ratio of the plastic rotation demands obtained from a specific analysis to the values of acceptance criteria prescribed by ASCE 41. The performance of each component is considered acceptable only if its DCR_N value is less than 1.0. The maximum DCR_N values resulting from the code-recommended pushover, the NMP, and the extended N2 procedures for all the beam and column elements at each story level are illustrated in Figs. 12 and 13, respectively. The mean DCR_N values obtained by the NDP (i.e. NL-RHA) as the most exact solution and the supplementary LDP verification are also superimposed in these figures. For the sake of brevity, the results are only presented for the CP performance level at the BSE-2 N hazard level. Note that, representing the results of the LDP analysis together with those obtained by the nonlinear procedures in a single plot does not denote their equivalence. The results demonstrate that in the entire example buildings, all the beams pass the assessment criteria of ASCE 41 for each of the analytical approaches. It can be seen in the figures that the enhanced pushover procedures provide acceptable estimates of the seismic performance in the beams for all the frames
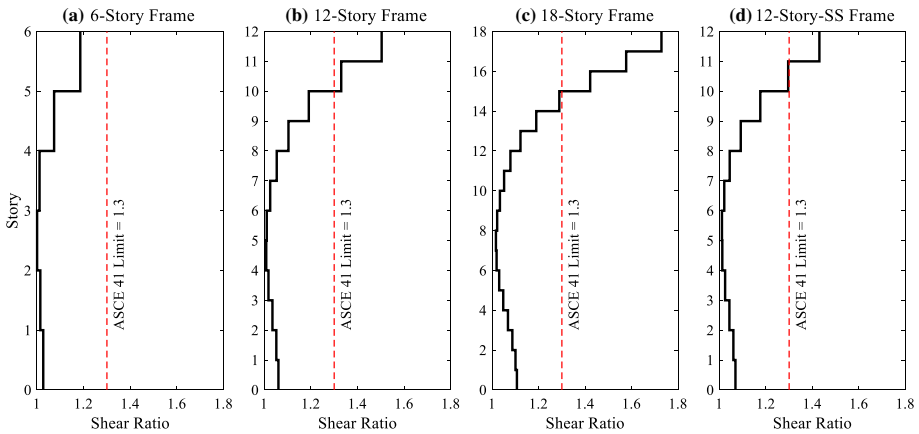
Fig. 11 The ratios of story shear resulting from the modal analysis considering modes required to obtain 90% mass participation to the corresponding story shear considering only the first mode response according to ASCE 41

even with the presence of the higher mode effects in the medium- and high-rise frames. Among these enhanced NSPs, the NMP procedure generally provides superior estimates of the beam DCR_N values, especially at the upper levels. In contrast to the enhanced NSPs, the code-recommended first-mode pushover method without the supplemental LDP evaluation fails to provide correct DCR_N values, by underestimating them at the upper part of the structures. When using the LDP procedure to supplement the Mode-1 NSP, excessively overestimated results at all story levels are obtained compared to the exact NDP.

Fig. 13 shows that all the columns successfully satisfy the acceptance criteria with a large margin of safety when evaluating using the nonlinear methods, i.e. the NSPs and NDP. It can be seen in the figure that the plastic mechanism only occurs at the base columns and DCR_N values for upper floors are almost zero. While all the NSPs predict the performance of the columns in the 6-story frame with sufficient accuracy, their results for taller buildings are not highly satisfactory. Among the enhanced pushover procedures, the results of the columns obtained by the NMP procedure are slightly closer to the NDP analysis. The results also demonstrate that the supplementary LDP procedure used for the frames with significant higher mode effects provides extremely large response quantities for columns in comparison to the benchmark analysis. As such, the DCR_N values of columns exceed the limiting value of 1.0 at some stories. Such overestimation is unacceptable and leads to the failure of the buildings to satisfy the acceptance criteria of ASCE 41 when the code-recommended NSP with its supplementary evaluation method is utilized for assessment. It can be inferred from the results that the pushover procedure proposed by ASCE 41 cannot provide a realistic estimation of the seismic performance even when its supplementary provisions are performed. On the other hand, the results of the enhanced pushover methods, especially the NMP procedure, are most satisfactory. Therefore, the implementation of enhanced pushover methods in the current seismic codes and guidelines would efficiently help to achieve a reasonable prediction of the seismic performance of structures.

From another perspective, the probabilistic approaches in the context of the second-generation performance-based earthquake engineering (Deierlein, Krawinkler, and Cornell 2003; FEMA 2006) have not been implemented in the design process for practical
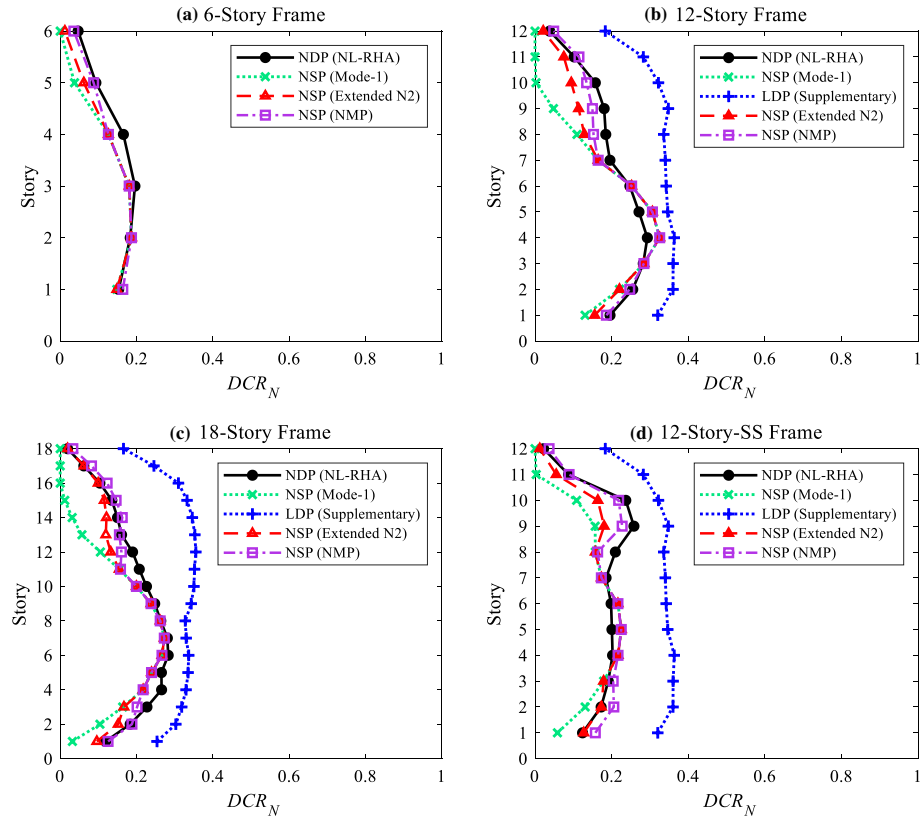
Fig. 12 Performance assessment results of the beam elements for the CP performance level at the BSE-2 N hazard level

purposes. A structure's fragility is defined as the probability of exceedance of an engineering demand parameter (EDP) at a given level of earthquake intensity, exceeding an associated capacity threshold C, which is a sign of violation of a given limit state (LS) (Bakalis and Vamvatsikos 2018). In order to define an analytical fragility function, a distribution function needs to be assumed. Previous research experience (Shome 1999; Cornell et al. 2002; Romão et al. 2011; M. Zarrin et al. 2018; Abyani et al. 2019; Mohamad Zarrin et al. 2020a, b) has revealed that the capacity and demand of various structural systems can be adequately approximated with the lognormal distribution function. By using the geometric mean and the standard deviation of maximum responses along the height of the structure (should be transferred to log space), determined based on the nonlinear response history analysis results, the lognormal distribution function for the EDP can be calculated (the so-called fragility function). When a single EDP is considered as the limit state capacity (e.g. the drift ratio of 0.02 as recommended by FEMA 273 (Council et al. 1997) for Immediate Occupancy limit state, and as the allowable design drift limit), the probability of exceedance of the limit state of interest can be determined by using this lognormal function. If the uncertainty in the limit state capacity is also considered, it can be treated as a random variable with a normal, lognormal, or non-normal (Ghasemi and Nowak 2017) distribution function. FEMA P-58 (Federal Emergency Management Agency (FEMA P 58-1) 2018)
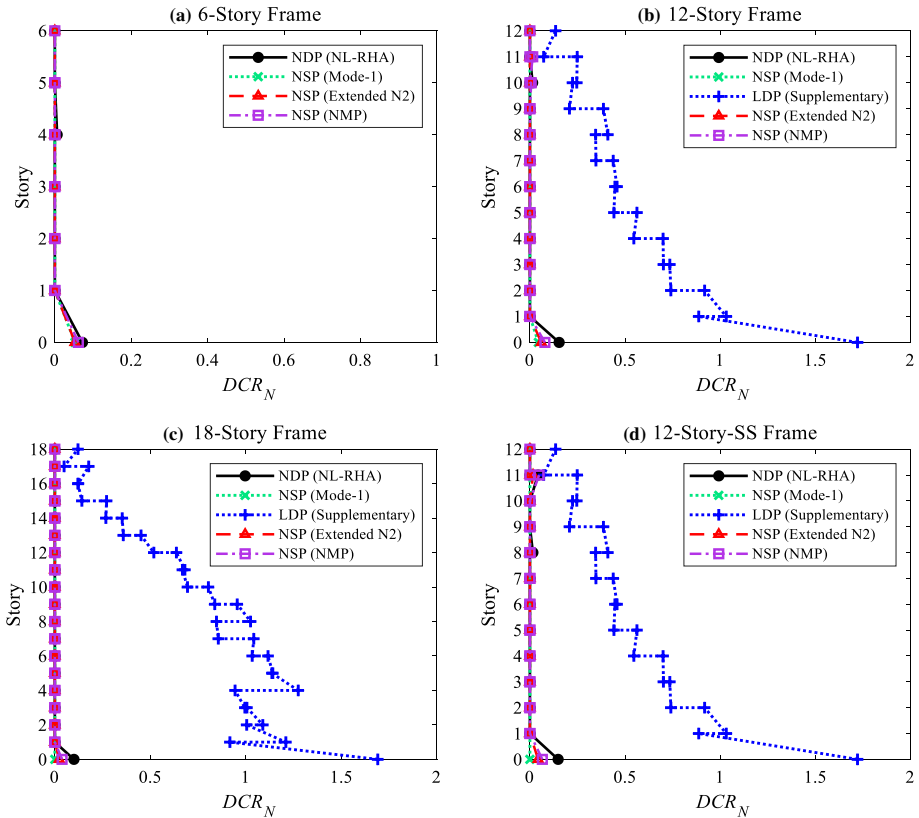
Fig. 13 Performance assessment results of the column elements for the CP performance level at the BSE-2 N hazard level

has guidelines for defining the dispersion of the limit states. In this case, both the demand and capacity have continuous distribution functions, and the probability of failure of the limit state can be computed in the context of the reliability analysis, thus requiring one more level of integration.

6 Conclusions

In this article, an alternative simplified pushover procedure is proposed to overcome the inherent drawbacks of conventional pushover procedures. The suggested method called the normalized multi-mode pushover (NMP) procedure is completely straightforward and avoids a burdensome process in the seismic evaluation of the structures. The NMP procedure takes the advantage of the concept of the extended N2 method (Kreslin and Fajfar 2011) in which the higher mode effects is considered using the elastic modal analysis. That is, it is assumed that the structure remains elastic when vibrating in higher modes. The application of the quadratic modal combination rules such as SRSS in the extended N2 method ignores the effects of sign reversal in the story displacements due to the contribution of higher modes. To tackle this problem inherent in the extended N2 method and

improve the accuracy of the predicted seismic demands, the NMP procedure employs the direct summation method to combine the peak modal responses pertaining to different vibration modes. In the NMP procedure, the ultimate responses of the structure are computed by modifying the results of a conventional pushover analysis such as the first-mode pushover analysis using the correction factors defined as the ratio of the normalized story drifts obtained by the elastic modal response analysis to the pushover analysis.

To validate the reliability of the proposed procedure in estimating seismic demands of building structures, it is applied to six steel moment-resisting frames with different heights. The verification is conducted at two earthquake intensity levels (i.e. the DBE and MCE levels) by evaluating the results of the NMP procedure with the average seismic demands resulting from the NL-RHA as the most accurate analysis method as well as two other non-linear static methods, namely first-mode pushover analysis, and the extended N2 method. The results are presented in terms of interstory drift ratios (IDRs), plastic hinge rotations, and the normalized demand-to-capacity ratio (DCR_N) of the components as a criterion for the seismic performance evaluation. The IDR results demonstrate that the NMP procedure can perfectly capture the results of the NL-RHA at the DBE level for almost all test buildings. At the higher seismic intensity level, the differences between the results of the proposed procedure and the benchmark solution are increased, but these differences are acceptable for a simplified analysis procedure nevertheless. On the other hand, the extended N2 method gives relatively better estimates of the IDRs at the MCE level in comparison to the DBE level, especially at upper floor levels. The remarkable superiority of the proposed procedure over the extended N2 in estimating the plastic rotations of the structural members can be observed in the results. In contrast to the IDRs, the plastic rotation demands obtained by the NMP procedure are in better agreement with the NL-RHA at the MCE level in comparison to the DBE level. The more acceptable predictions of the local demands by the NMP procedure compared to the extended N2 method is because of the preservation of the sign of modal responses when computing the final responses of the structure. A similar conclusion can be drawn for the DCR_N values of the components used to evaluate the performance of the structures based on ASCE 41-17. More importantly, it was shown that the enhanced NSPs provide a notably better prediction of the seismic performance of the buildings than the code-recommended first-mode-based NSP. Further, the supplementary evaluation procedure prescribed by ASCE 41 for the buildings with significant higher mode effects to overcome the limitations of the first-mode pushover analysis fails to provide a reasonable prediction of structural performance, by grossly exaggerating the DCR_N values for both the beam and column elements. For this reason, the authors suggest the implementation of the enhanced pushover procedures in the current seismic codes in lieu of the code-recommended NSP.

Declarations

Conflict of interest The authors declare that there is no conflict of interests regarding the publication of this paper.

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