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Response modification factor for steel moment-resisting frames by different pushover analysis methods

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ABSTRACT

The earthquake loads imposed to the structures are generally much more than what they are designed for. This reduction of design loads by seismic codes is through the application of response modification factor (*R*-factor). During moderate to severe earthquakes, structures usually behave inelastically, and therefore inelastic analysis is required for design. Inelastic dynamic analysis is time consuming and interpretation of its results demands high level of expertise. Pushover analysis, recently commonly used, is however, a simple way of estimating inelastic response of structures. Despite its capabilities, conventional pushover analysis (CPA) does not account for higher mode effects and member stiffness changes. Adaptive pushover analysis (APA) method however, overcomes these drawbacks. This research deals with derivation and comparison of some seismic demand parameters such as ductility based reduction factor, R_μ, overstrength factor, Ω, and in particular, response modification factor, R, from capacity curves obtained from different methods of APA and CPA. Three steel moment-resisting frames of 3, 9 and 20 stories adopted from SAC steel project are analyzed. In pushover analyses for each frame, eight different constant as well as adaptive lateral load patterns are used. Among the main conclusions drawn is that the maximum relative difference for response modification factors was about 16% obtained by the methods of conventional and adaptive pushover analyses.

1. Introduction

Previously elastic analysis was the main tool in seismic design of structures. However, behavior of structures during recent earthquakes indicates that relying on just elastic analysis is not sufficient. On the other hand, nonlinear dynamic analysis, although yields accurate results, is time consuming and at times complex. Such analysis must be repeated for a group of acceleration time histories, not to mention the need for delicate interpretation of its results. Researchers have long been interested in developing fast and efficient methods to simulate nonlinear behavior of structures under earthquake loads. The idea of inelastic static pushover analysis was first introduced in 1975 by Freeman for single degree of freedom (SDOF) systems. Then other researchers extended this method for multi-degree of freedom systems [1–4]. Conventional pushover analysis (CPA), despite its strengths, has some drawbacks. For example, the shape of lateral load pattern stays the same during analysis. This shape is usually based on the first elastic mode of the structure. In other words, the higher mode effects or the role of more effective modes are not accounted for. The latter may be the source of significant errors in seismic response evaluation of tall buildings. Therefore, Moghadam and Tso [5] and later Chopra and Goel [6] introduced multi-mode methods to overcome this problem. The most applicable method among them is modal pushover analysis (MPA) in which the structure under a load pattern corresponding to the elastic mode shapes is pushed to a certain lateral displacement. Then the results obtained for each mode are combined using SRSS or CQC methods. Another drawback that is common in both CPA as well as MPA is the lack of accounting for the change in member and/or global stiffness matrices at subsequent steps of analysis. In each step plastic hinges form and the structure further goes in inelastic range, followed by a reduction in structural global stiffness. However, the load pattern is still kept based on the original stiffness and elastic mode shapes. In other words, the lateral load pattern is not in conformance with the reduced stiffness. Bracci et al. [7], Sasaki et al. [8], Satyarno et al. [9], Matsumori et al. [10], Gupta and Kunnath [11], Reqena and Ayala [12], and Elnashai [13] proposed different ways of conforming the loading pattern with the structural stiffness. The method of adaptive pushover analysis (APA) was first developed in 2004 by Antoniou and Pinho [14]. Not only is this method multi-mode based, but also, the lateral loading pattern is adapted according to the changes in stiffness matrix at each step of the analysis. Following the Northridge earthquake in 1994, the seismic design provisions of design and material codes such as ASCE, UBC, AISC and ACI codes fundamentally

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changed. Equivalent static method in most seismic design codes is based on the use of response modification factor, R (or sometimes called force reduction factor). In fact design loads are obtained by reducing/dividing the earthquake loads by the R factor. By reducing the earthquake loads the structure will enter into inelastic range. Therefore, in order to dissipate the earthquake energy, the structure will have to experience rather large inelastic deformations. The structural capacity in withstanding the earthquake loads is related to its capacity in deforming in inelastic range, or its ductility capacity. For a system with idealized bilinear behavior (see Fig. 1), structural ductility, μ is defined as the ratio of maximum displacement to the displacement corresponding to the yielding point. Structures with higher force reduction factor, R, require higher ductility capacity, μ. Therefore, R and μ factors are interrelated and play important role in energy dissipation mechanism of the structures.

In order to overcome the shortcomings of the CPA, Antoniou and Pinho proposed two different methods for adaptive pushover analyses; force-based adaptive pushover analysis (FAPA) [14] and displacement based adaptive pushover analysis (DAPA) [15].

2. Force-based adaptive pushover analysis (FAPA)

In any adaptive pushover analysis, in each step, the software updates the lateral loading. In FAPA the updating algorithm includes four parts: 1) defining nominal lateral load vector and floor inertial mass; 2) derivation of load factor; 3) derivation of normalized load vector applicable to the structure; and 4) updating the load vector. The first part happens only once in the beginning of analysis. The other three parts repeat for each step in FAPA analysis. The load pattern vector is automatically obtained and updated according to the above algorithm. The nominal force vector P_0 is defined uniformly along the height. The distribution of load along the height at each step is through the normalized force vector $\bar{F}$, which is derived based on dynamic characteristics of the structure for that step and the elastic response spectra of the given earthquake. Therefore, the floor inertial masses are also required. The load vector P at each step is obtained by multiplying the nominal load vector P_0 by the load factor λ for that step (Eq. (1)). The load factor λ depends on the type of analysis (load control or response control) and the number of steps. In other words, the management of lateral load increase is by application of this factor.

$$P = \lambda \cdot P_0 \tag{1}$$

Normalized load vector $\bar{F}$, computed in the beginning of each step, provides the shape of increasing load vector at each step. Any stiffness changes must be reflected in this vector. Therefore, at each step, by solving eigenvalue problem, the mode shapes and mode participation factors are derived. Floor loads at each step, then, would be:

$$F_{ij} = \Gamma_j \varphi_{ij} M_i \tag{2}$$

where i is the floor number, j is the mode number, Γ_j is the j^{th} mode participation factor, ϕ_{ij} is the j^{th} mode value at the i^{th} floor, and M_i is the mass at the i^{th} floor. Eq. (2) provides floor load corresponding to unit response acceleration. For a given mode j, with known frequency ω_j or period T_j, spectral acceleration $S_{a,j}$ would be available and Eq. (2), changes as below:

$$F_{ij} = \Gamma_j \varphi_{ij} M_i S_{a,j} \tag{3}$$

To obtain the value of floor load, F_i, for a given floor number i, the floor loads corresponding to different modes are combined using SRSS or CQC methods. Therefore, at each step, there would be one single load pattern. Since the shape of load pattern is important and not its magnitude, the load values for each floor are normalized by the total value, i.e., the sum of all the floor loads in that step:

$$\bar{F}_i = \frac{F_i}{\sum_{i=1}^{N} F_i}. \tag{4}$$

Having known $\bar{F}_t$, λ_t and $\Delta\lambda_t$ at any analysis step t, and P_0, the adaptive load vector P_t can be obtained using either of the below equations relating to incremental or total updating.

$$P_t = P_{t-1} + \Delta\lambda_t \cdot \bar{F}_t \cdot P_0 \tag{5}$$

$$P_t = \lambda_t \cdot \bar{F}_t \cdot P_0 \tag{6}$$

where P_{t-1} is the adaptive load vector for the previous step. Researchers showed that the results obtained using FAPA method may be at times erroneous. This could be attributed to the use of SRSS method for combining the modal floor loads. Since irrespective of the sign of modal values, they always participate in the combined floor load with a positive sign, and this may discount the higher mode effects [15,16]. Therefore, Antoniou and Pinho introduced displacement based adaptive pushover (DAPA) method [15] that is to overcome the aforementioned issues.

3. Displacement based adaptive pushover analysis (DAPA)

The proposed algorithm at each step contains four parts: 1) defining nominal lateral load vector U_0 and floor inertial mass; 2) derivation of

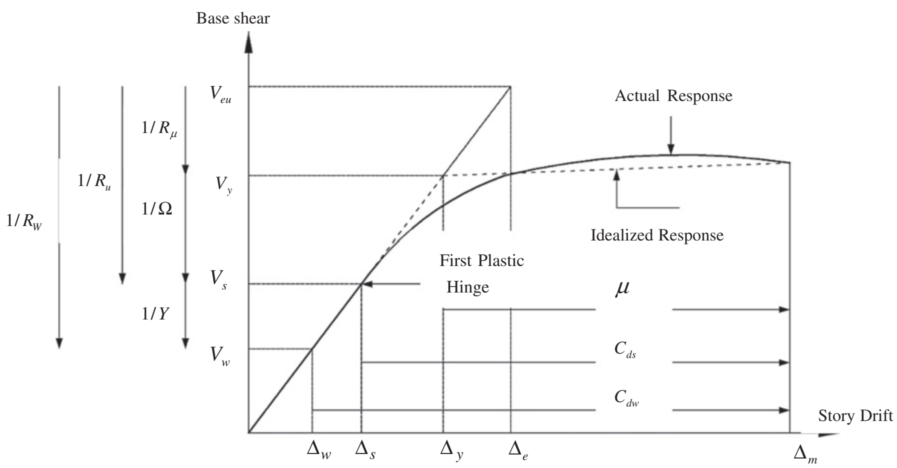

Fig. 1. Capacity curve for a structure along with its bilinear idealization in pursuit of seismic demand parameters [18].

load factor; 3) derivation of normalized load vector applicable to the structure; and 4) updating the displacement vector. The first two parts are similar to FAPA method, except that the load vector is now displacement based. The load vector U is defined as:

$$U = \lambda \cdot U_0 \tag{7}$$

The normalized $\bar{D}$ vector, computed in the beginning of each step or at the end of last step represents the shape of the load vector (or the increase in load vector). At the end of each step (after application of each load increment) an eigenvalue problem is solved and depending on the current stiffness of the system, the mode shapes and participation factors are derived. Modal floor loads can be combined by SRSS or CQC methods. Normalization of load vector in DAPA method is either based on the story displacement or the interstory displacement. Both are explained in the following.

3.1. Normalization based on story displacement

The load vector, in this method, can be obtained directly from modal analysis (Eq. (8)). This is similar to FAPA method, except that instead of force components, the story modal displacements are combined using SRSS method [15].

$$D_i = \sqrt{\sum_{j=1}^{n} D_{ij}^2} = \sqrt{\sum_{j=1}^{n} \left(\Gamma_j \cdot \varphi_{ij}\right)^2} \tag{8}$$

D_{ij} is the i^{th} floor displacement due to the j^{th} mode.

3.2. Normalization based on interstory displacement

Since maximum interstory displacement better describes the level of damage during an earthquake than maximum story displacement, Antoniou and Pinho [14] proposed the following method of normalizing lateral load vector that is based on interstory displacement. It is assumed that floor displacement D_i at each floor level i is the sum of interstory displacements below that level. Also, it is assumed that interstory displacement at level i is the SRSS combination of modal interstory displacements [15].

$$D_i = \sum_{k=1}^{i} \Delta_k \quad \text{with} \quad \Delta_i = \sqrt{\sum_{j=1}^{n} \Delta_{ij}^2} = \sqrt{\sum_{j=1}^{n} \left[\Gamma_j \left(\varphi_{i,j} - \varphi_{i-1,j}\right)\right]^2} \tag{9}$$

For a given earthquake and/or design response spectrum Eq. (9) turn into Eq. (10) where $S_{d,j}$ is the j^{th} modal displacement.

$$D_i = \sum_{k=1}^{i} \Delta_k \quad \text{with} \quad \Delta_i = {}^{i}\sqrt{\sum_{j=1}^{n} \Delta_{ij}^2} = \sqrt{\sum_{j=1}^{n} \left[\Gamma_j \left(\varphi_{i,j} - \varphi_{i-1,j}\right) S_{d,j}\right]^2} \tag{10}$$

Although results show an improvement using the latter method for derivation of load vector, this is still an approximate method, because of the assumed simultaneous occurrence of maximum interstory displacements, while this is not usually the case. However, due to the improvement of the results this method is adopted as standard DAPA analysis method in this research [15]. The final shape of the load pattern in DAPA method is taken from the shape of normalized floor displacements. Normalized floor displacement $\bar{D}_i$ is obtained by dividing the i^{th} floor displacement by the maximum floor displacement.

$$\bar{D}_i = \frac{D_i}{\max D_i} \tag{11}$$

Having derived normalized vector $\bar{D}_i$; primary nominal load vector U_0, load factor λ_t and/or incremental load factor $\Delta\lambda_t$, the adaptive load vector U_t at any step t of the DAPA analysis can be updated following one of the equations, Eq. (12) or (13). U_{t-1} is the adaptive load vector in the previous step $t-1$.

$$U_t = U_{t-1} + \Delta\lambda_t \cdot \bar{D}_t \cdot U_0 \tag{12}$$

$$U_t = \lambda_t \cdot \bar{D}_t \cdot U_0 \tag{13}$$

4. Response modification factor (*R* factor)

Researchers have so far proposed different methodologies for derivation of R factor. These methods in general, fall into two main groups: the European and the American methods. In this study one of the most important American methods, so-called Uang method, is adopted. The parameters used in Uang method, illustrated in Fig. 1, are defined in the following [17,18].

Fig. 1 depicts variation of structural base shear versus story total drift in a typical pushover analysis. This curve is idealized as the response of bilinear elasto-plastic system in pursuit of seismic demand parameters including R factor.

4.1. Global ductility of the structure

Global ductility ratio of the structure, μ_s is defined as ratio of maximum lateral displacement (Δ_{max}) to lateral displacement at yield (Δ_y).

$$\mu_S = \frac{\Delta_{max}}{\Delta_y} \tag{14}$$

4.2. Ductility based force reduction factor (R_μ)

Structures with ductility capacity can dissipate hysteretic energy of earthquakes. Therefore, maximum elastic earthquake force (base shear, V_{eu}) can be reduced to structural general yield strength (V_y) at collapse occurrence. Ductility based reduction factor R_μ can then be defined as

$$R_\mu = \frac{V_{eu}}{V_y}. \tag{15}$$

4.3. Overstrength factor (Ω)

Overstrength factor represents the reserved strength in the structures between the general yield point (V_y) and the formation of the first plastic hinge (V_s).

$$\Omega = \frac{V_y}{V_s} \tag{16}$$

4.4. Allowable stress factor (Y)

Depending on the definition of design stresses (allowable or ultimate stress) in different design codes (ASD and LRFD), the Y factors could have different values. In general, the allowable stress factor Y is defined as the ratio of structural strength (base shear) at formation of the first plastic hinge (V_s) to the strength corresponding to allowable design stresses (V_w).

$$Y = \frac{V_s}{V_w} \tag{17}$$

Table 1
First three modal periods of structures (s).

Building	Mode number		
	1	2	3
SAC-3	0.379	0.125	0.080
SAC-9	1.008	0.385	0.225
SAC-20	1.656	0.588	0.343

Table 2
Component yield criteria for different members in steel moment resisting frames [22].

Rotation at yield	Expected flexural strength	Member
$\theta_y=\frac{ZF_{ye}L_b}{6EI_b}$	$M_{CE}=ZF_{ye}$	Beams
$\theta_y=\frac{ZF_{ye}L_c}{6EI_c}\left(1-\frac{P}{P_{ye}}\right)$	$M_{CE}=1.18ZF_{ye}\left(1-\frac{P}{P_{ye}}\right)\leq ZF_{ye}$	Columns

F_{ye} = expected yield strength of the material, L_b = beam length, L_c = column height, M_{CE} = expected flexural strength, P = axial force at target displacement in pushover analysis, P_{ye} = expected axial yield force of the member, θ_y = yield rotation and Z = plastic section modulus.

Therefore, response modification factor (R-factor) in seismic codes allowing ASD (allowable stress design) method would be:

$$R=\frac{V_{eu}}{V_w}=\frac{V_{eu}}{V_y}\frac{V_y}{V_s}\frac{V_s}{V_w}=R_\mu\Omega Y. \tag{18}$$

5. Structural models

Following the objectives in this research, three buildings of 3, 9, and 20 stories, previously designed and studied by SAC steel project [19,20] are used. These models hereafter are called SAC-3, SAC-9 and SAC-20. The lateral load resisting system in these buildings is moment-resisting frames that are located in the perimeter of the buildings. The building site is in Los Angeles, CA, USA and the design code is UBC-1994 [19]. Geometric properties of the SAC frames are illustrated in Fig. 11 and their first three modal periods are given in Table 1. The Seismostruct software is used to perform all pushover analyses [21]. This software takes advantage of fiber elements that are capable of accounting for material nonlinearity. The $P\Delta$ effect is considered in the analyses. The steel properties are selected similar to the original study, i.e., yield stress for beams F_y = 36 ksi (248 MPa) and for columns = 50 ksi (345 MPa) and modulus of elasticity, E_s = 29,000 ksi (200,000 MPa). Nonlinear behavior of steel is assumed to be bilinear with 3% strain hardening.

6. Performance criteria

The yield criteria for steel beams and columns, applicable for pushover analysis, are taken from ASCE 41-06 [22]. Table 2 depicts component yield criteria for different members in steel moment resisting frames [22].

The output of any pushover analysis is the variation of base shear with lateral displacement. The criteria as to where the ultimate capacity of the building structure is arrived and the analysis is completed are twofold: 1) formation of collapse and/or mechanism, i.e., where the structure cannot take any more lateral load; and 2) the arrival of an interstory drift limit usually set by the seismic codes. In the seismic design code of Iran, Standard 2800 [23], for structure with a fundamental period $T\leq 0.7$ s, this limit is 3.57% and for $T>0.7$ s, it is 2.85%. For a given performance level, this lateral target displacement is the maximum lateral displacement the structure likely experiences during the design earthquake for the given hazard level. The capacity (pushover) curve is, of course, sensitive to the degradation and post-peak negative stiffness. According to section 3.3.3.2.5 of ASCE 41.06 [22] this displacement shall be the calculated target displacement or the displacement corresponding to the maximum base shear whichever is least. This approach is consistent with the primary life safety performance objective of seismic regulations of model building codes [26]. Several methods for determination of target displacement have been proposed in literature and associated with structural performance levels. In the present study the allowable displacement for the design earthquake of Iranian seismic code has been considered.

7. Derivation of response modification factor, R, by different pushover analysis methods

The strategy in this paper for derivation of seismic demand parameters such as R factor is to first obtain the capacity and/or pushover curves for the given structure. Then by bilinear idealization of such curve as per Fig. 1, the demand parameters can be derived.

Three main types of pushover analysis are performed in this study: 1) conventional pushover analysis (CPA), 2) force based adaptive pushover analysis (FAPA), and 3) displacement based adaptive pushover analysis (DAPA).

In CPA the load pattern is kept constant throughout the analysis. Two constant load patterns considered in this study are: 1) UNIFORM load pattern in which for a given floor, the lateral floor load is proportional to its mass/weight, and 2) MODAL load pattern, where the lateral floor load is proportional to the story shear distribution calculated by SRSS combination of modal responses (Seismic design code of Iran, Standard 2800 [23]). Response spectrum analysis of the structures was performed by the ETABS software [24].

The $S_{d,j}$ and $S_{a,j}$ used in this study respectively for DAPA and FAPA, are 5% damped response spectra of three major earthquakes: Northridge 1994, Tabas 1978 and Imperial Valley 1940. Table 3 shows the characteristics of these ground motions. In all modal and adaptive pushover analyses the first 10 modes are used. For each building eight pushover analyses are performed: two CPA (with constant load patterns of Uniform and Modal), three FAPA (for three mentioned earthquakes), and three DAPA (for the three earthquakes).

Load vector in each analysis step t can be obtained by total updating or incremental updating [14]. Incremental updating is considered in all numerical computations of the models. This is performed in accordance with Eqs. (5) and (12). Load factor increment ($\Delta\lambda_t$) in each step was determined according to loading or solution schemes including “load control” and “response control”. The response control scheme was employed in FAPA and CPA analyses and the load control scheme in DAPA analysis. The parameters P_0 equal to 100 (kN) and U_0 equal to 0.01 m was selected and total number of loading steps was assumed equal to 500. Load factor increment ($\Delta\lambda_t$) in DAPA analysis was equal to ((1/500)∗(allowable roof displacement for design earthquake adopted by Iranian seismic code in meter)) and in FAPA

Table 3
Earthquake characteristics from PEER site database.

Earthquake name, date	Station	Component	PGA (g)	PGV (cm/s)	PGD (cm)
Tabas, 1978/09/16	9101TABAS	TAB-LN	0.836	97.8	36.92
Northridge, 1994/01/17	24278 Castaic-Ridge Route	ORR090	0.568	52.1	4.21
Imperial Valley, 1940/05/19	117 El Centro Array # 9	I-ELC180	0.313	29.8	13.32

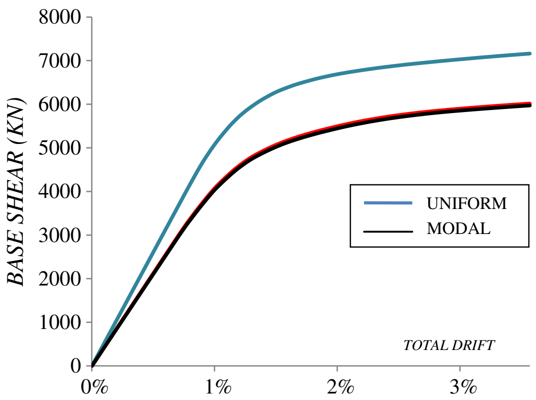

Fig. 2. CPA pushover/capacity curves for SAC-3 structure for two load pattern, and drift limit of 3.57%.

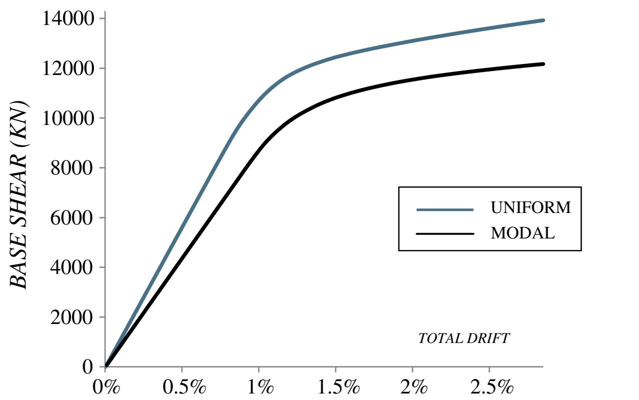

Fig. 5. CPA pushover/capacity curves for SAC-9 structure for two load pattern, and drift limit of 2.85%.

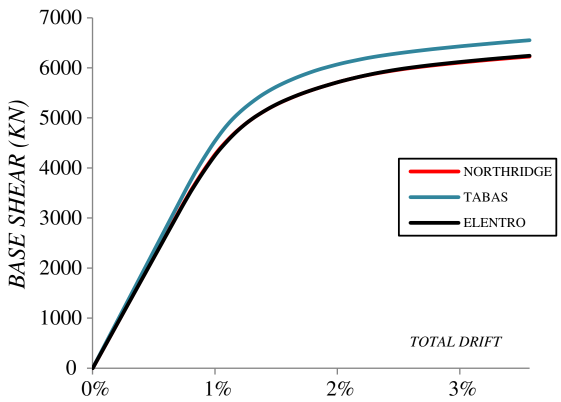

Fig. 3. FAPA pushover/capacity curves for SAC-3 structure for three earthquake spectra, and drift limit of 3.57%.

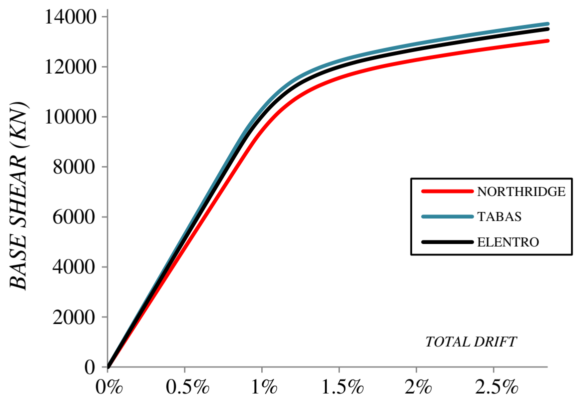

Fig. 6. FAPA pushover/capacity curves for SAC-9 structure for three earthquake spectra, and drift limit of 2.85%.

and CPA analyses was calculated by software for attainment of a determined response displacement increment in a controlled node.

Figs. 2 to 10 illustrate the results for these analyses for the three structural models of SAC-3, SAC-9, and SAC-20. The locations of plastic hinges are depicted in Fig. 11 at the arrival of interstory drift limit set by the seismic design code of Iran. The FAPA and DAPA results in this figure are based on Northridge 1994 earthquake.

The bilinear idealization of the pushover curves shown in Figs. 2 to 10 (Fig. 1) was performed following the criteria set by ASCE 41-06 [22] by the MATLAB software.

8. Results and discussion

Tables 4 to 6 list all seismic parameters shown in Fig. 1 from eight different pushover analyses required in derivation of ductility ratio, μ, overstrength factor, Ω, for SAC-3, SAC-9, and SAC-20 structures, respectively.

In derivation of R_μ, the maximum elastic base shear, V_{eu} is required. Subjected to a given acceleration time history and assuming elastic behavior for the structure, the maximum base shear recorded would be V_{eu}. Miranda and Bertero [25], having conducted a large parametric study on nonlinear response of SDOF systems to different

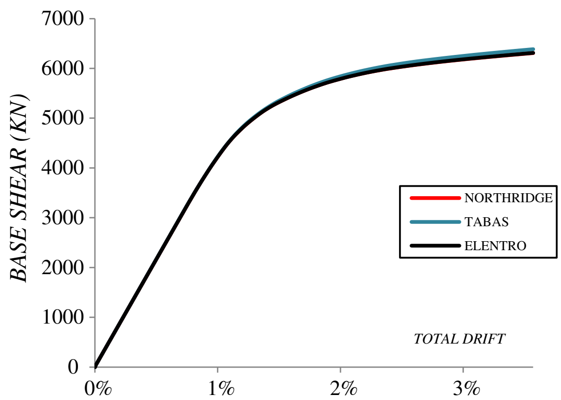

Fig. 4. DAPA pushover/capacity curves for SAC-3 structure for three earthquake spectra, and drift limit of 3.57%.

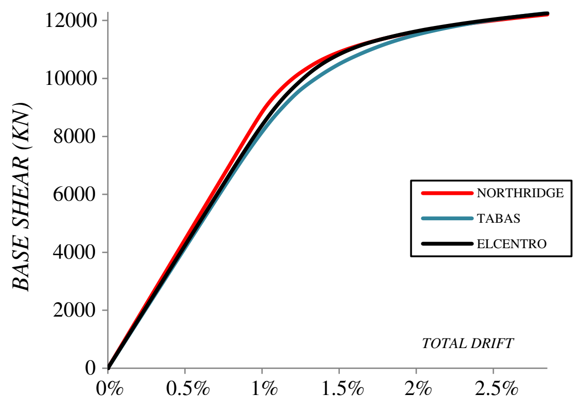

Fig. 7. DAPA pushover/capacity curves for SAC-9 structure for three earthquake spectra, and drift limit of 2.85%.

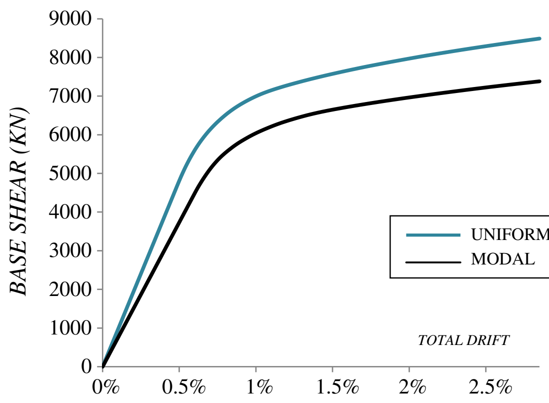

Fig. 8. CPA pushover/capacity curves for SAC-20 structure for two load patterns, and drift limit of 2.85%.

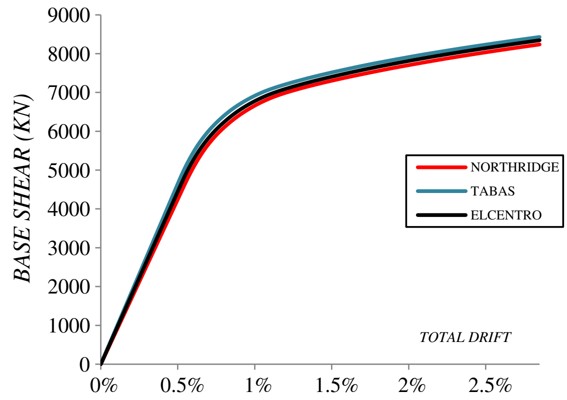

Fig. 9. FAPA pushover/capacity curves for SAC-20 structure for three earthquake spectra, and drift limit of 2.85%.

earthquake records on different sites, proposed the following empirical relationship for R_μ.

$$R_\mu = \frac{\mu - 1}{\Phi} + 1 \quad (20)$$

μ is the global ductility ratio and Φ as defined below (for sediment foundation soils) is a function of μ and the fundamental period of the structure, T.

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} e^{-2(\ln(T) - 0.2)^2} \quad (21)$$

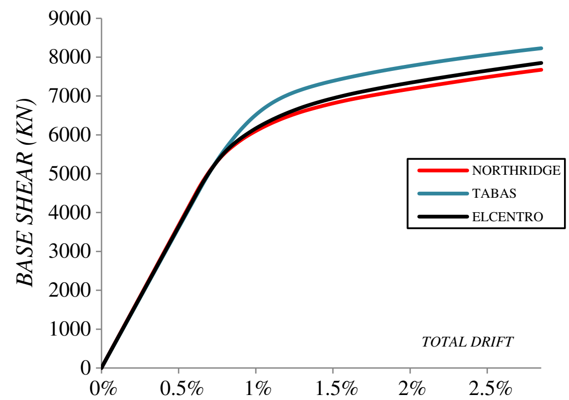

Fig. 10. DAPA pushover/capacity curves for SAC-20 structure for three earthquake spectra, and drift limit of 2.85%.

Having obtained R_μ and assuming a design allowable stress factor of $Y = 1.5$ [18], the R factors for all structures and under all pushover types are derived and listed in Tables 7 to 9.

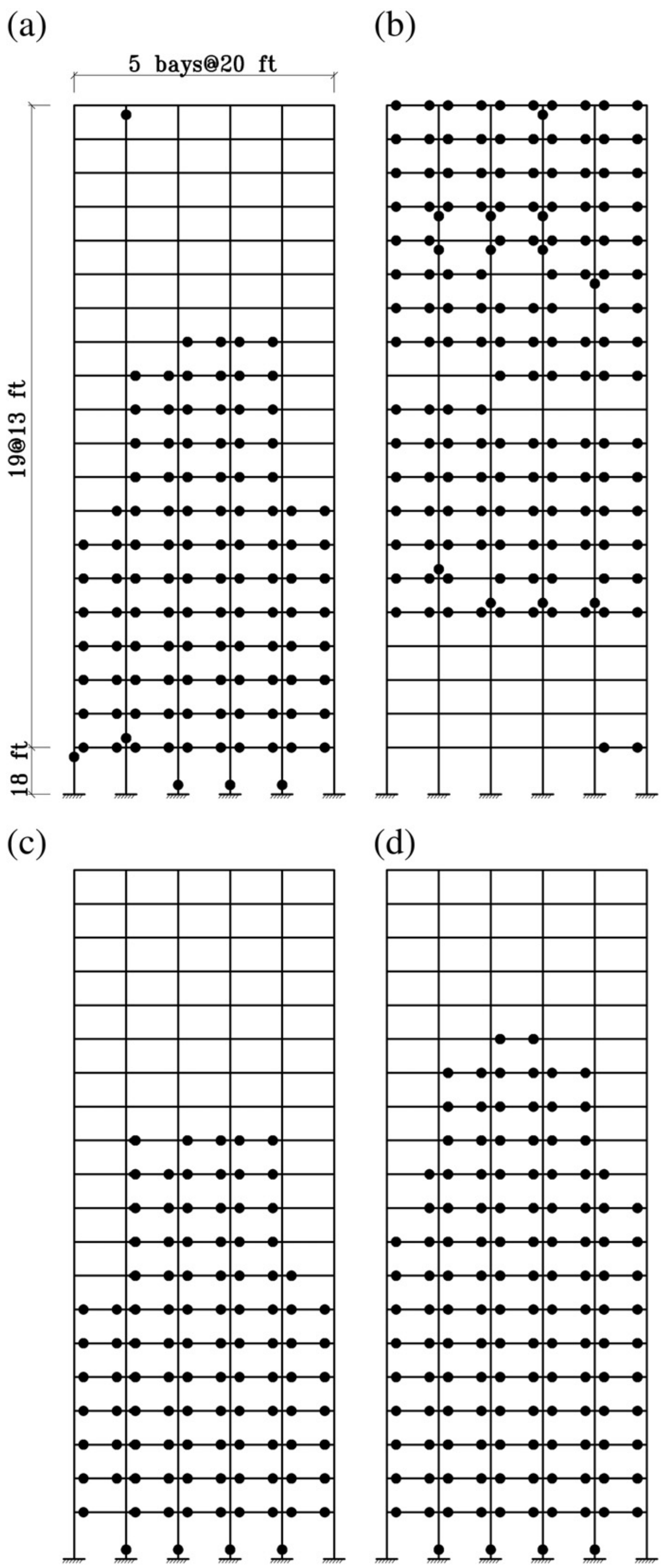

Fig. 11. Frame geometries and locations of plastic hinges at arrival of code drift limit, a. DAPA with $S_{a,j}$ from Northridge earthquake, b. FAPA with $S_{a,j}$ from Northridge earthquake, c. CPA with uniform load pattern, d. CPA with modal load pattern (1 ft = 0.3048 m).

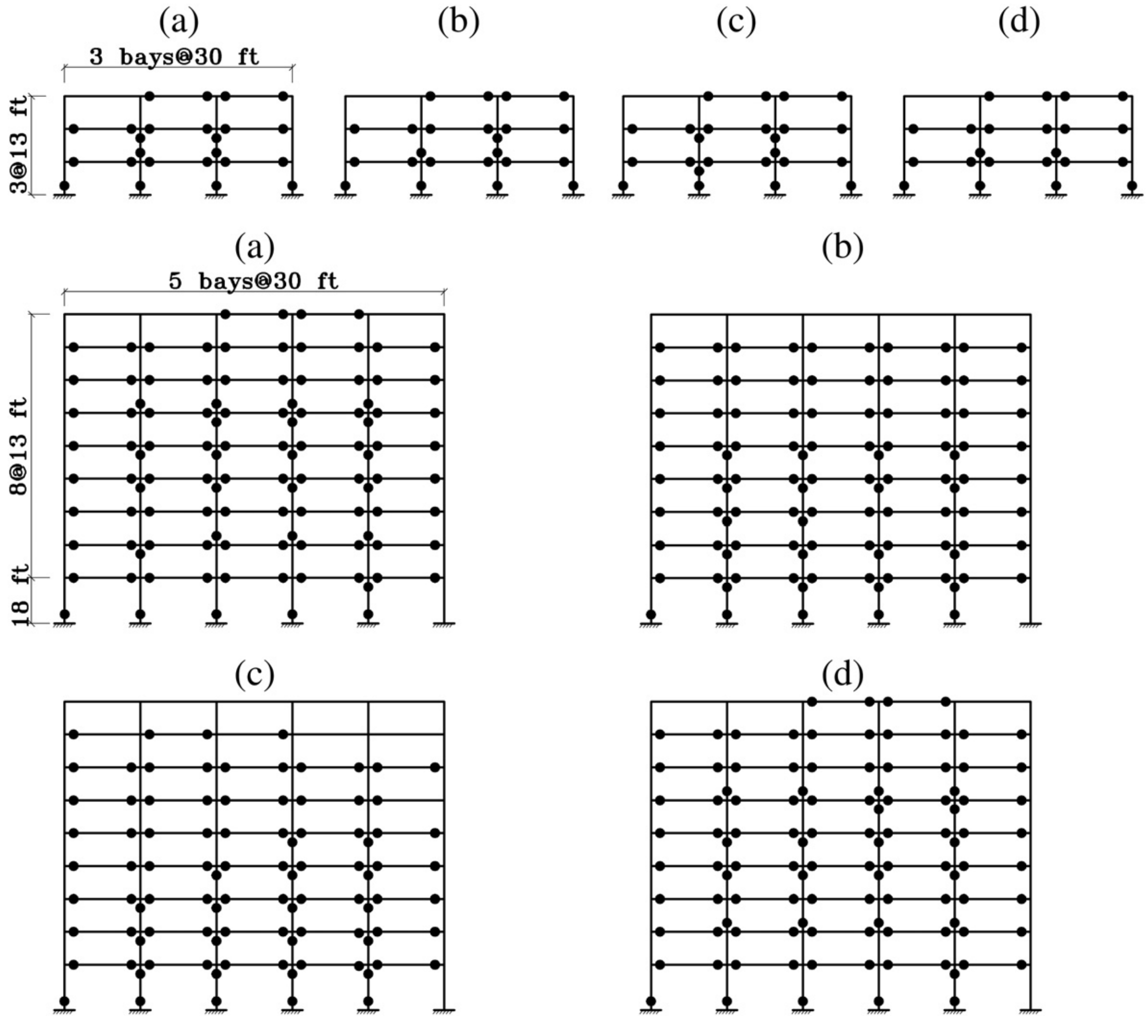

Fig. 11 (*continued*).

For each building structure, depending on the number of earthquakes or the number of constant load patterns, there are more than one or two *R* factors. ASCE 41-06 recommends using the smaller *R* factor. This is because the smaller *R* factor will lead to a larger design base shear and/or a safer design. Table 10 provides a short/final list of the *R* factors for the three structures used in this study.

9. Summary and conclusions

Depending on the severity of the design earthquake, the structures may undergo nonlinear behavior. Nonlinear dynamic analysis, although yields accurate results, is time consuming and at times complex. Researchers have long been interested in developing fast and efficient methods to simulate nonlinear behavior of structures under earthquake loads. Conventional pushover analysis (CPA), despite its strengths, has some drawbacks. For example, the shape of lateral load patterns is constant and stays the same during analysis. This shape is usually based on the first elastic mode of the structure. In other words, the higher mode effects or the role of more effective modes are not accounted for. Later modal pushover analysis (MPA) was introduced which accounts for higher mode effects. A common drawback in both CPA as well as MPA is the lack of accounting for the change in member and/or global stiffness matrices during pushover analysis. Adaptive pushover analysis (APA) was therefore developed in 2004 by Antoniou and Pinho, which not only is multi-mode based, but also, the changes in stiffness matrix at each step of the

Table 4
Results of different pushover analyses, ductility ratios, μ, and overstrength factors, Ω, for SAC-3.

Load pattern	Δ_s (cm)	Δ_y (cm)	Δ_{max} (cm)	V_s (kN)	V_y (kN)	V_{max} (kN)	Ω	μ
CPA, uniform	7.56	14.57	39.48	3305.1	6376	7107	1.93	2.71
CPA, modal	8.41	14.02	32.94	2968.8	4948	5794	1.67	2.35
DAPA, Northridge	8.15	15.00	38.71	2965.4	5455	6239	1.84	2.58
DAPA, Tabas	8.23	15.00	37.18	3000.8	5461	6283	1.82	2.48
DAPA, El Centro	8.15	15.00	37.61	2965.0	5454	6222	1.84	2.51
FAPA, Northridge	8.24	14.01	34.89	3069.3	5221	6088	1.70	2.49
FAPA, Tabas	7.98	14.43	37.53	3138.5	5676	6464	1.81	2.60
FAPA, El Centro	8.24	14.15	34.98	3053.7	5247	6102	1.72	2.47

Table 5
Results of different pushover analyses, ductility ratios, μ, and overstrength factors, Ω, for SAC-9.

Load pattern	Δ_s (cm)	Δ_y (cm)	Δ_{max} (cm)	V_s (kN)	V_y (kN)	V_{max} (kN)	Ω	μ
CPA, uniform	18.70	38.25	60.56	5613.8	11,495	12,635	2.05	1.58
CPA, modal	23.37	44.98	77.56	5463.6	10,519	11,622	1.92	1.72
DAPA, Northridge	22.95	44.62	77.77	5464.1	10,624	11,695	1.94	1.74
DAPA, Tabas	24.20	43.91	70.12	5392.8	9775	11,330	1.81	1.60
DAPA, El Centro	24.22	47.67	88.4	5506.6	10,788	11,947	1.97	1.85
FAPA, Northridge	21.46	42.85	66.72	5488.1	10,967	12,028	2.00	1.56
FAPA, Tabas	19.76	40.37	62.26	5612.3	11,477	12,505	2.04	1.54
FAPA, El Centro	20.19	40.73	63.11	5549.7	11,207	12,313	2.02	1.55

Table 6
Results of different pushover analyses, ductility ratios, μ, and overstrength factors, Ω, for SAC-20.

Load patterns	Δ_s (cm)	Δ_y (cm)	Δ_{max} (cm)	V_s (kN)	V_y (kN)	V_{max} (kN)	Ω	μ
CPA, uniform	27.23	53.08	95.54	3235.9	6308	7250	1.95	1.80
CPA, modal	32.31	63.08	113.54	2978.5	5816	6570	1.95	1.80
DAPA, Northridge	33.69	66.16	126.93	3057.8	6002	6871	1.96	1.92
DAPA, Tabas	36.92	73.85	118.62	3292.7	6583	7354	2.00	1.61
DAPA, El Centro	34.62	68.46	139.39	3113.2	6156	7139	1.98	2.04
FAPA, Northridge	30.00	58.46	100.20	3145.3	6130	7035	1.95	1.71
FAPA, Tabas	28.15	54.62	97.39	3231.4	6269	7213	1.94	1.78
FAPA, El Centro	28.62	56.16	97.84	3142.2	6167	7101	1.96	1.74

Table 7
R_μ and R factors for different pushover analyses for SAC-3 structure.

	CPA, uniform	CPA, modal	DAPA, Northridge	DAPA, Tabas	DAPA, El Centro	FAPA, Northridge	FAPA, Tabas	FAPA, El Centro
R_μ	2.41	2.12	2.30	2.22	2.25	2.23	2.32	2.21
R	6.98	5.31	6.35	6.06	6.21	5.69	6.30	5.71

Table 8
R_μ and R factors for different pushover analyses for SAC-9 structure.

	CPA, uniform	CPA, modal	DAPA, Northridge	DAPA, Tabas	DAPA, El Centro	FAPA, Northridge	FAPA, Tabas	FAPA, El Centro
R_μ	1.80	1.99	2.02	1.83	2.17	1.77	1.74	1.76
R	5.54	5.54	5.88	4.97	6.41	5.31	5.32	5.33

analysis are accounted for. Antoniou and Pinho (2004) later introduced two different versions of APA; namely, force based adaptive pushover analysis (FAPA) and displacement based adaptive pushover analysis (DAPA) methods. Using different constant and adaptive load patterns in pushover analysis methods, this study dealt with derivation of seismic demand parameters for steel moment-resisting. Among the main conclusions drawn are:

1) R factors obtained by the methods of conventional (CPA) and adaptive (FAPA or DAPA) pushover analyses tend to be different. The maximum relative difference for response modification factors was about 16% due to larger results in adaptive pushover considering different seismic records in Tables 7, 8 and 9.
2) Ductility ratios (μ) obtained by the methods of conventional and adaptive pushover analyses tend to be different. The maximum relative difference in ductility ratios was about 17% due to larger results in adaptive pushover. 3) Displacement based adaptive pushover analyses (DAPA) yield higher inelastic lateral displacements and/or ductility ratios compared to the other pushover methods. 4) For high-rise and mid-rise buildings (SAC-20 and SAC-9) the different shapes of constant load pattern in CPA result in close R factors. 5) The use of different earthquake response spectra for high-rise and mid-rise buildings in FAPA method does not have considerable effect on the R factors and related parameters.

Table 9
R_μ and R factors for different pushover analyses for SAC-20 structure.

	CPA, uniform	CPA, modal	DAPA, Northridge	DAPA, Tabas	DAPA, El Centro	FAPA, Northridge	FAPA, Tabas	FAPA, El Centro
R_μ	1.93	1.93	2.07	1.71	2.21	1.83	1.91	1.86
R	5.65	5.65	6.09	5.13	6.56	5.35	5.56	5.47

Table 10
Response modification factor, R from different pushover analysis methods.

Structural models	CPA	FAPA	DAPA
SAC-3	5.31	5.71	6.06
SAC-9	5.54	5.31	4.97
SAC-20	5.65	5.35	5.13
Average	5.5	5.45	5.39

The results' confidence can be improved by more analytic models with other assumptions in numerical computations such as method of maximum lateral displacement computation and period dependent relations for ductility based force reduction factor (R_μ).

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