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Research Paper

Comparative Evaluation of Effects of Lateral Load Distribution Pattern and Performance Point Determination Methods

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Abstract

Nonlinear static analysis makes a significant contribution to earthquake engineering research by demonstrating the capacity of the underlying loading structure and reducing computational effort in comparison with nonlinear dynamic analysis. This analysis involves two main steps: the distribution of lateral load and the determination of the performance point. The accuracy of the results of this analysis is directly related to how these two stages are carried out. In this study, three methods of lateral load pattern (traditional, modal, and adaptive) and four procedures for determining the performance point based on the capacity spectrum (with special attention to the energy capacity spectrum) have been evaluated. The accuracy of these methods in estimating the displacement performance of regular and irregular masses in reinforced concrete frames of 5, 10, and 15 stories was compared to the results of dynamic analysis using 25 far-field records. The analysis results demonstrate the superiority of adaptive load patterns over traditional and modal patterns, particularly in tall and irregular frames. Additionally, the energy spectrum method, known for its simpler process, yields more accurate results.

Keywords: Reinforced Concrete Frames; Irregular; Energy Spectrum

1. Introduction

The criteria for structural member failure are typically defined in terms of displacement and strain, using force-based analytical methods. These force or resistance criteria are easily applicable when the structure behaves linearly or when linear force-displacement models are available. However, during earthquakes, most force and resistance criteria cannot be used to control displacement- and strain-based mechanisms [1]. Various methods have been proposed to evaluate the performance of structural displacement, among which nonlinear dynamic time history analysis is considered the best method. However, issues such as the complexity of the analysis process, the extended duration of dynamic analysis, and the challenges in selecting appropriate records have led to the use of alternative methods such as nonlinear static analysis (pushover).

Pushover analysis involves the application of a designated lateral load pattern and a step-by-step increase to achieve the target displacement of the structure or complete collapse. It is a straightforward approach to estimate the structural strength in the plastic range and can help identify the weak areas of the structure [2].

The determination of structure displacement capabilities using nonlinear static analysis generally consists of two parts:

A) Applying lateral loads with a specified load pattern and determining the pushover curve (base shear-displacement) for the structure

B) Determining the target displacement of the structure in relation l0 to the pushover curve.

Performing each of these parts has a direct impact on the accuracy of the results of the analysis.

However, pushover analysis has fundamental theoretical weaknesses arising from its initial assumption regarding the dependence of the nonlinear response of the structure on the response of an equivalent single-degree-of-freedom system, the short duration of vibration, and ignoring the effects of the earthquake. Therefore, due to the aforementioned limitations, pushover analysis requires proper validation compared to nonlinear dynamic analysis [3].

1.1. Lateral load patterns

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Lateral load patterns are generally divided into constant and variable types. In the constant lateral load pattern mode, the inertial forces are assumed to be constant and unchanged during the earthquake. Obviously, this assumption is unrealistic and approximate, and given this type of load pattern, it is not possible to take into account in the analysis the changes in the distribution of lateral inertial forces caused by stiffness changes resulting from nonlinear behavior. However, many regulations accept approximate analysis to avoid complexity and recommend constant lateral load patterns.

In recent years, a number of researchers have suggested using an adaptive pushover method in which an adaptive load pattern is employed. Thus, at each loading stage, the shape of the load pattern is modified and adapted according to the modal characteristics of the structure. In the conventional pushover method, where the structure is pushed in a single step with a constant load pattern to the target displacement, the structure is pushed in the adaptive pushover analysis in several steps with a new load pattern in each step (calculated using the modal characteristics of the structure in the same step) until the target displacement is reached [4].

In the conventional pushover method, the structure is pushed in a single step with a constant load pattern to the target displacement. In contrast, in adaptive pushover analysis, the structure is pushed in several steps, with a new load pattern in each step (calculated using the modal characteristics of the structure at that step) until the target displacement is reached [4].

In 1998, Krawinkler and Seneviratna conducted a study on nonlinear static analysis and introduced this method of analysis as a suitable method for engineering judgment in estimating structural seismic performance. This study mentioned suitable pushover characteristics to obtain proper insight into the structural aspects that control the structure during an earthquake, relatively accurate prediction of some displacement results such as the need for nonlinear deformation for structures that behave in accordance with the main modes of vibration, and some structural weaknesses that may be hidden in static analysis, such as story mechanisms, unauthorized deformations, resistive irregularities, and so on. Moreover, the existing weaknesses in pushover analysis were evaluated, and the inaccuracy of the deformation results obtained from the analysis for structures where higher modes had considerable effects on the vibration of the structure and the relationship of shear and relative displacement of stories were highly dependent on the load pattern. This problem can be partly mitigated by using multiple load patterns instead of a single one [5].

In 1996, Paret et al. addressed the issue of higher vibration modes for the first time and concluded that regulatory methods were not capable enough of estimating the potential of higher vibration modes in high-period structures. Therefore, they introduced the modal criticality index to determine the contribution of each mode. They introduced and proposed the simple yet practical multimodal pushover procedure [6].

In 2002, Moghaddam and Tso introduced a modified modal pushover method called the pushover result combination. This method obtains the maximum seismic response * by combining * pushover analysis results with load patterns consisting of several predefined modes related to various modes of vibration. Final structural responses are obtained through the weighted combination of the results of different pushovers [7]. A similar approach based on the multimodal method is the modal pushover analysis proposed by Chopra and Goel in 2002. It involves idealizing pushovers according to different modes, defining the target displacement of each * separately, and determining the final response by combining * model responses through the square-root-of-sum-of-squares [8].

In 1997, Brassi et al. were the first to propose a method using a fully adaptive model. In this method, the analysis starts with an initial assumption of the load pattern (usually triangular), while the lateral load is subsequently increased and calculated from the instantaneous base shear and story resistances of the previous load step [9].

In 2000, Gupta and Kunnath proposed a different method of adaptive pushover, in which the applied load was continuously adapted based on the dynamic characteristics of the structure and a structure-related response spectrum was used to define the load pattern. In this method, eigenvalue analysis was performed to determine the instantaneous stiffness of the structure before each load increase [10].

Elnashai in 2001 proposed an adaptive pushover method which, with the one-step analysis algorithm, has all of the advantages mentioned in previous studies such as adaptation, multimodality, and considering the reduction of system resistance and length of structural period by adapting the load distribution in each step (or specified steps). The dynamic properties of the structure are obtained by using the average results of the analysis of the eigenvalues in the modes concerned, which take into account the instantaneous stiffness of the structure at each step of the analysis. The site response spectrum (or response spectrum of any desired record) was used directly to scale the forces and calculate the probability of resonance between earthquake vibration and vibration of each mode. This force-based method paved the way for many other studies and many efforts were made to increase its accuracy and stability[11].

Tajik et al. conducted a study in 2016 to improve the existing adaptive pushover methods. Because these methods use second-degree combination methods to combine the effects of different modes, the direction of the mode shape vector is ignored. Tajik et al. presented a direct vector combining process in which the relative contribution and vector direction of each mode are taken into account. This direct combination method was applied with adaptive displacement pushovers, and the comparative results of the analyses showed an increase in the accuracy of the adaptive pushovers using this combination [12].

In 2016, Shaynfar et al. introduced the energy-based adaptive pushover analysis (EAPA), inspired by the two methods of force-based adaptive pushover (FAP) and story shear-based adaptive pushover (SSAP). To calculate input energy, the method uses the work done in each story at each step of lateral load pattern adaptation. They tested their proposed method on three types of steel moment frames with different heights and evaluated three parameters: relative story drift, maximum shear of stories, and maximum turning moment. Their results show the accuracy of this method for steel frames compared with FAP and SSAP.

In 2017, with the aim of considering the effects of higher modes on the nonlinear static analysis of tall structures, Vafaei and Safari proposed a modal pushover method in which each vibration mode was assigned a participation factor based on the modal shear distribution. This participation factor can specify the importance of each mode in determining the shape of the lateral load pattern [13].

In 2017, Esfahanian and Aghakuchak introduced a dynamic load pattern (DLP) pushover approach to incorporate the effect of near-fault earthquakes in nonlinear static analysis. The most important feature of this method, which makes it simple, is its determination of a non-invariant load pattern that takes into account the influence of all parameters based on averaging the forces of the stories arising from linear time history analysis under a set of records. In this method, the final seismic demands of the pushover are obtained from the results of traditional pushover analysis under the load pattern of the first mode of vibration and the DLP dynamic model [14].

To consider the effect of higher modes in pushover analysis, Amini and Poursha proposed a force-based modal adaptive load distribution model in 2018, which is an adaptive model of the modal load distribution method. The method comprises a combination of several adaptive analyses, including an adaptive pushover analysis with the first mode of vibration plus one or two modal adaptive pushover analyses applied to the structure via an advanced load distribution derived from the algebraic sum of the modal forces of the stories. The final results are obtained from the outputs of the first-mode pushover analysis and the modal pushover analyses. A comparison of the results of this method with other modal and adaptive load distributions showed that this method was more accurate in estimating the displacement demands of structures, especially in the higher stories [15].

In 2018, Hadi et al. proposed a story shear-based pushover method which, instead of using acceleration or displacement of stories as the basis of load pattern adaptation, calculates the relative shear of stories, adapts the structure at each step based on the dynamic characteristics, and converts them into equivalent lateral forces. A comparison of this method with the previously proposed adaptive load pattern and dynamic time history analysis showed that the proposed adaptive shear-based model has similar precision to previous models, but it is more compatible with existing commercial software [16].

Rahmani et al. proposed a nonlinear static analysis process called upper bound adaptive pushover in 2019 to estimate the seismic demands of tall structures. It took into account both the impact of higher modes and changes in the dynamic properties of structures. In the proposed pushover process, the applied lateral load is calculated using the upper bound method with the first two modes of vibration and adapted at each step of the analysis. The accuracy of this method in the analysis of four steel moment frames was compared with other dynamic pushover and time history analytical methods, suggesting the greater accuracy of the proposed method in estimating the relative displacement of stories as well as its higher performance in predicting the formation points of plastic joints along the structure's height [17].

1.2 Determination performance point of structures

Estimating seismic displacement demands and the most nonlinear structural deformation during earthquakes is a key aspect of performance-based structural design that has been studied extensively (Fajfar, 2000 [18]; Panagiotakos & Fardis, 2001 [19]; Sullivan et al., 2006 [20]). Some of these studies were also presented in the form of standards such as ATC-40 (1996), FEMA 356 (2000), and FEMA 440 (2006).

Freeman et al. conducted a case study of the seismic risks of existing structures in 1975, proposing the basis of the capacity spectrum as a rapid method for estimating the seismic demands of structures [21]. In cooperation with the Applied Technology Council (ATC), Freeman subsequently developed the capacity spectrum method as a practical guideline.

The method of displacement coefficients was presented in FEMA 356 and FEMA 440 to determine the target displacement of a structure using a set of displacement modification and periodic factors, each of which is determined through tables based on the type of structural system, lateral resistance system, number of stories, and height of the structures.

The N2 method was first proposed by Fajfar in the mid-1990s based on the Q model introduced in 1981 by Susan and Saeedi. This method is very similar to the ATC-40 capacity spectrum method mentioned earlier. Their main difference is that, unlike the capacity spectrum method which uses the elastic demand spectrum, this method uses the inelastic demand spectrum [22].

Housner first used the concept of energy balance in 1956 to calculate the average amount of energy absorbed by a structure in design concepts [23].

In addition, Hernandez et al. (2004) [24] and Mezzi et al. (2006) introduced the concept of energy as an alternative index to obtain the capacity curve [25]. In 2008, Leelataviwat et al. [26] used an applied method of the structure energy balance equation to determine the performance point, which is described in detail as follows.

In 2013, Masoumi and Monavari introduced an energy-based method of cyclic loading to determine the performance of concrete structures. In this study, 13 concrete structures were subjected to static and cyclic loading, and structural failure was controlled under cyclic loading. Additionally, the FEMA 356 displacement coefficients method was used as a verification criterion for the introduced approach. One of the advantages of the proposed method is its ease of use, as well as its incorporation of both concepts of capacity and structural demand. They showed that the greatest damage to structures occurs at 50% to 80% of the structure's height, while the roof displacement of the structure is usually considered to determine the performance point, which requires more attention [27].

n 2013, Seyed Bahram Beheshti and Jafar Kamali replaced the pseudo-velocity spectrum with effective peak velocity (EPV) with the aim of modifying the energy capacity spectrum equation. This was done to reduce the effects of the highs and lows on the pseudo-velocity spectrum, which showed acceptable effects on the accuracy of the results.

In 2023, Abdollahzadeh, Farahani, and Amirchoupani conducted research comparing the mean target displacement obtained from direct time history analysis using the self-centering flag-shaped hysteresis model with the proposed equation to illustrate the accuracy of the methodology.

The verification analysis showed that the suggested equation could estimate the target displacement of self-centering structural systems with an error of less than 10% [28].

Additionally, in 2024, Abdollahzadeh, Farahani, and Amirchoupani determined the damage-based inelastic displacement ratio (IDR) for fully self-centering (SC) systems with flag-shaped hysteresis behavior under pulse-like loading. This approach can predict the target displacement compared to those obtained from direct time history analysis, where the difference between the estimated and mean target displacements is below 15% [29].

2. Method of Modal Combinations

All the mentioned attempts to incorporate the effects of higher modes in nonlinear static analysis were based on the idea that the effects of higher modes are combined to estimate the overall performance from the independent results. One of the most interesting ways to influence vibrational modes in nonlinear static analysis was proposed by Matsumori et al. in 1999, who applied these modes directly through the addition and subtraction of different modes. Therefore, to better estimate the seismic demand, they used two load patterns: one derived from the algebraic sum of the modal shear of stories and the other derived from the algebraic difference of the modal shear of stories [30].

In 2004, Kunnath thoroughly investigated the Method of Modal Combinations (MMC) and proposed Equation 1 to determine the lateral force of each story in load distribution[31].

$$F_j = \sum_{i=1}^{n} \Gamma_i m_j \Phi_{ji} S_{ai}(\xi_i, T_i) \tag{1}$$

Where F_j is the share of the force of the j-th story from the lateral load pattern distribution, m_j is the mass of the class j-th story, Φ_{ji} is the modal shape of j-th story in the i-th mode, $S_{\alpha i}$ is the spectral acceleration corresponding to the structural period in the i-th mode, and Γ_i is the participation factor of the i-th mode, obtained from Equation 2.

$$\Gamma_i = \frac{\sum m_j \Phi_{ji}}{\sum m_j \Phi_{ji}^2} \tag{2}$$

Kunnath also analyzed an 8-story concrete structure using the load pattern in accordance with the first and second modes of vibration and extracted two pushover analyses from the displacement results. He also analyzed the structure under his proposed load pattern once using the sum of the effects of the first and second modes and once using the difference of the effects of the two primary modes, and extracted two pushover analyses based on the displacement results. Finally, the results of the pushover analysis were compared with the time history output of the dynamic analysis, as shown in Figure 1.

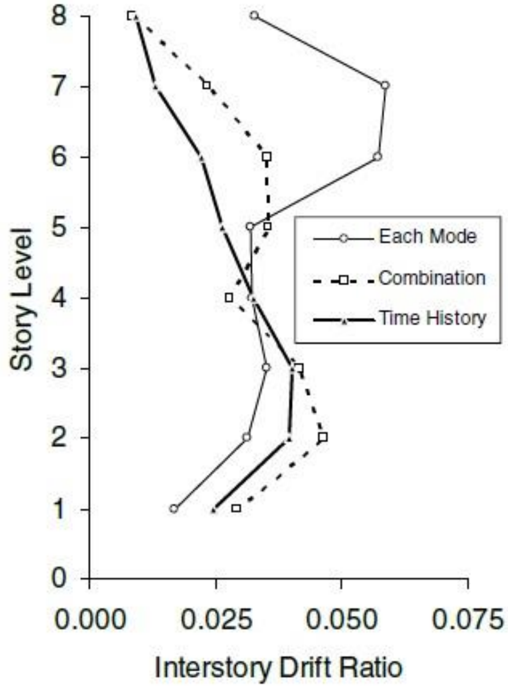


Fig. 1. Comparison of the displacement results of various load patterns with the output of the time history analysis[32]

As shown in Fig. 1, the displacement results of pushover analyses of the load patterns in accordance with the first and second modes yield very conservative results in the lower and upper stories separately, while the displacement results of the modal pushover analyses are more consistent with the outputs of time history analysis in both low and high stories.

To illustrate the validity of Kunnath's proposed modal combination method, Banihashmi analyzed a 6-story system under modal pushover, multimodal, and dynamic nonlinear time history analyses. The results showed good convergence of the multimodal combination (MMC) method. The outputs of time history analysis differed from those of other pushover analysis methods arising from different modes and modal pushover analysis (MPA) [32].

3. Adaptive Pushover Analysis

In 2004, Antoniou and Pinho investigated the benefits and limitations of traditional and adaptive pushovers and developed Elnashai's (2001) proposed adaptive pushover method. In this study, the pushover analysis algorithm was practically improved and applied in SeismoStruct software.

The proposed method consisted of two types of force-based lateral load and displacement patterns as follows:

- Force-based Adaptive Pushover (FAP) analysis
- Displacement-based Adaptive Pushover (DAP) analysis

The steps of Antoniou and Pinho's proposed comparative pushover method are described in four main steps:

1. Define an initial nominal load vector and the mass inertia moment
2. Calculating the load factor
3. Calculating the normally scaled modal vector
4. Adapting the loading vector

The first step is performed only once, while the next three steps are repeated at each step of the analysis [33].

The shape of the loading vector is determined at each loading step using the existing vector loading algorithm. In order for the loading shape at each stage not to deviate from the corresponding dynamic response profile of the structure, the nominal vector P_0 must always be in the form of a uniform distribution in (rectangular) height. The magnitude of the load vector P at each step of the analysis is determined by multiplying the nominal vector P_0 and the load factor λ, as follows in Equation 3:

$$P = \lambda P_0 \tag{3}$$

The load coefficient value λ is automatically increased using the load control tool until it reaches a predetermined target response or results in numerical instability. To determine the vector shape at each step of the analysis, the normalized scaled modal F-vector, calculated at the beginning of each loading step, is used.

$$F_{ij} = \Gamma_j \Phi_{ij} M_i \tag{4}$$

Where M_i is the mass of i-th story.

4. General Principles of Capacity Spectrum Method

The principle of any earthquake-resistant design approach is based on a comparison between two quantities: capacity and demand. These two quantities can relate to force, displacement, stiffness, energy, or any other quantity that needs to be controlled. In any case, the equation should hold true: (Capacity > Demand)

In capacity-based approaches, these two terms are represented by two curves, where capacity depends on the features of the structure and demand depends on the parameters of extreme ground motion. In fact, the structural capacity curve and the earthquake demand curve are plotted in a coordinate system and compared to determine whether or not the structural capacity is responsive to earthquake demand. Therefore, the behavior of the structure indicates the degree to which its capacity responds to seismic demand.

4.1. ATC40 Capacity Spectrum Method

The performance point is obtained in ATC-40 using the capacity spectrum method. In this method, the capacity of a structure is identified by its capacity curve obtained through incremental (pushover) nonlinear static analysis. The seismic demand also comes from the probable earthquake spectrum of the site. These two curves are then transformed into spectral acceleration versus spectral displacement curves and plotted simultaneously in a Cartesian coordinate system. By applying a reduction to the seismic demand spectrum due to the nonlinear effects on the behavior of the structure, a point is finally obtained as the performance point of the structure. This point represents the maximum possible displacement of the structure due to an earthquake. By obtaining this point and comparing it to the limit provided by the regulations, one can determine the functional level of the target structure during a particular earthquake [34].

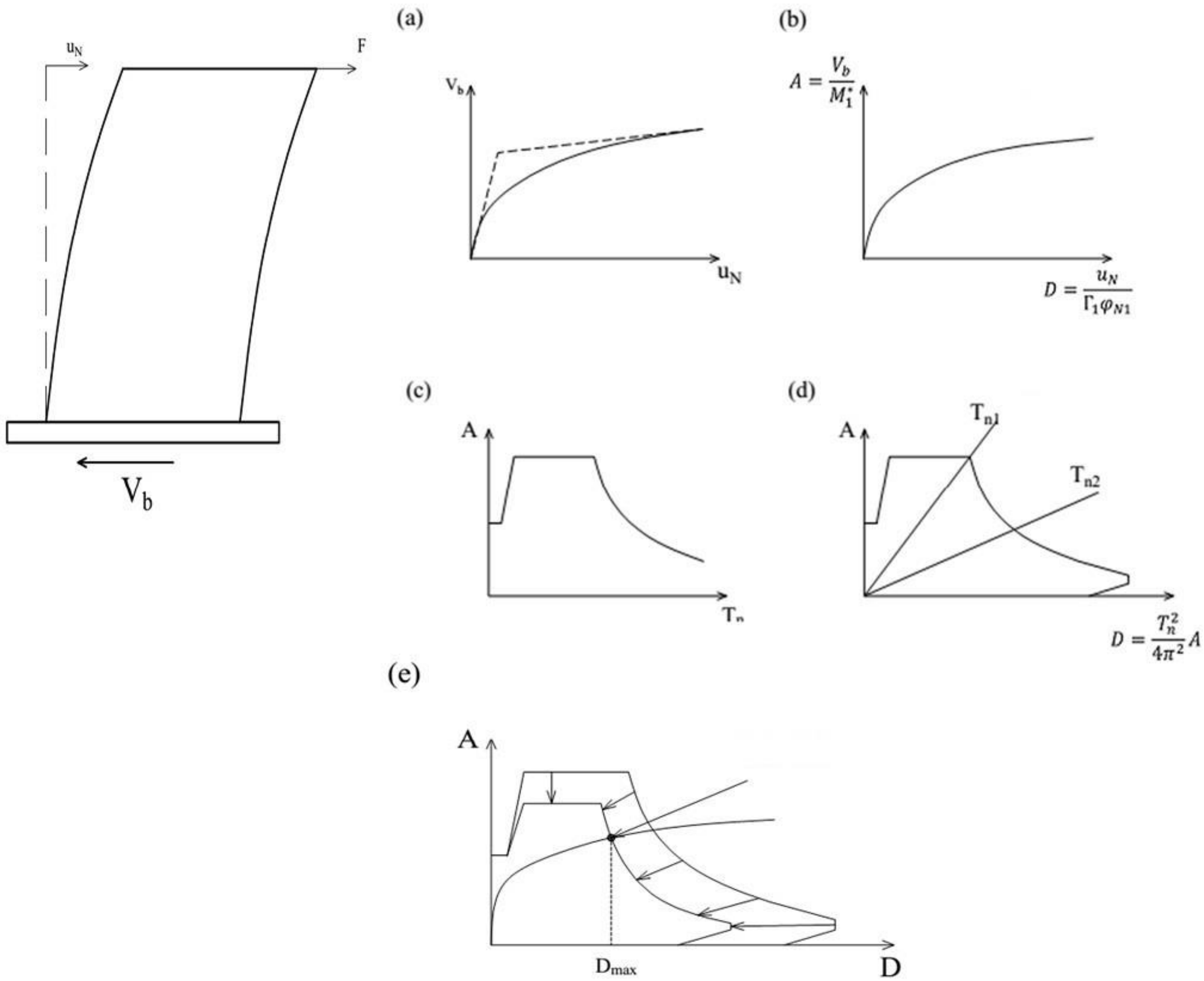


Fig. 2. (a) Pushover curve (b) Capacity spectrum (c) Acceleration-period response spectrum (d) Demand spectrum (e) Performance point[33]

4.2. N2 Capacity Spectrum Method

This method combines the pushover analysis of a multi-degree-of-freedom model whit the straight-spectrum analysis of an equivalent single-degree model. The elastic response spectrum of the site is transformed into the non-elastic spectrum using equations (5) and (6), and the intersection point of the bilinear capacity graph and the non-elastic response spectrum coordinates determines the performance point for a single-degree-of-freedom system [35].

$$S_a = \frac{S_{ae}}{R_\mu} \tag{5}$$

$$S_d = \frac{\mu}{R_\mu} S_{de} = \frac{\mu}{R_\mu}\frac{T^2}{4\pi^2} S_{ae} = \mu \frac{T^2}{4\pi^2} S_a \tag{6}$$

Where S_a and S_d are the acceleration and non-elastic spectral displacements, respectively; Sae and Sde are the elastic and spectral displacements, respectively; μ represents the ductility coefficient, defined as the ratio of maximum displacement to yield displacement. R_μ is the resistance reduction factor due to ductility, and T represents the period of the structure.

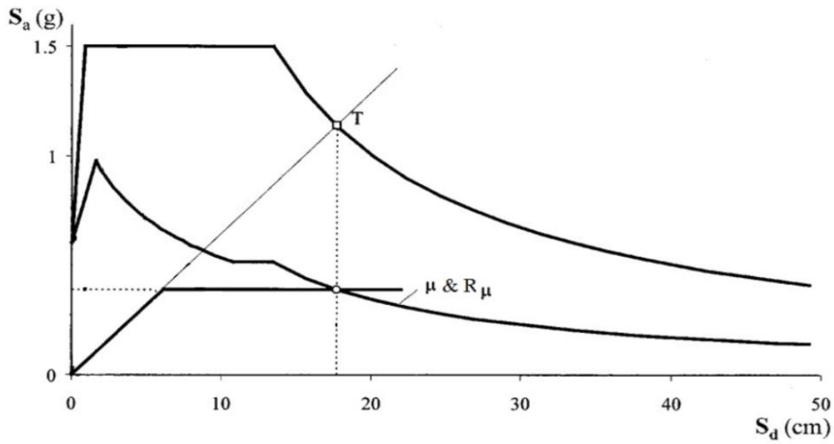


Fig. 3. Determination of the performance point at the intersection point of the plastic response spectrum and the capacity spectrum[21]

4.3. FEMA440 Equivalent Linearization Method

The FEMA440 equivalent linearization method is an extended form of the ATC40 capacity spectrum method. The commonality of these two methods lies in the determination of the capacity curve of the structure using the pushover diagram, as well as determination of parameters such as damping, initial period and the need for μ ductility. However, to reduce the elastic response spectrum for estimating the seismic demand of an equivalent one-degree-of-freedom system, the effective period of T_{eff} and the effective damping of β_{eff} are used and attempts are made to overcome the weaknesses of the proposed ATC40 method[36].

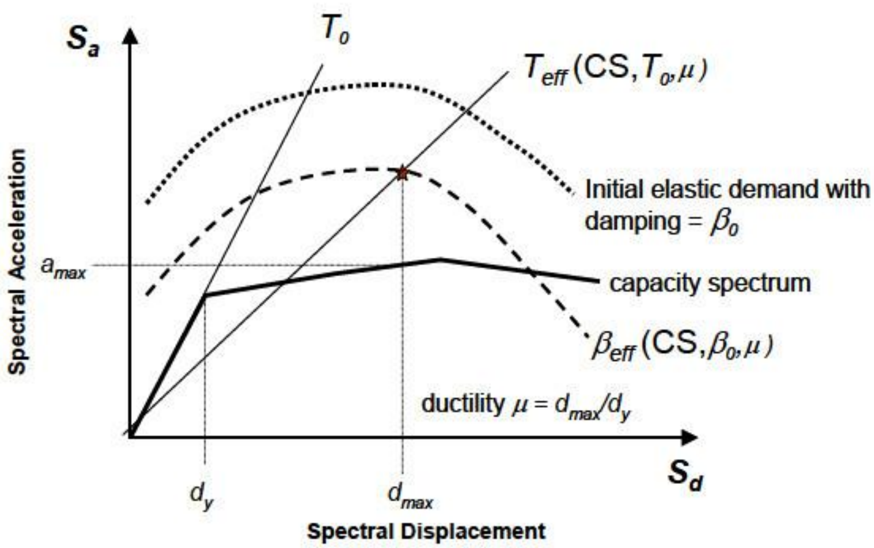


Fig. 4. The general process of determining the performance point using the equivalent linearization method[34]

4.4. Energy Capacity Spectrum Method

In the performance-based plastic design method, the design base shear for a particular risk level is obtained by equalizing the work required to push the structure uniformly to achieve the target displacement and the energy required by an elastoplastic single-degree-of-freedom system to achieve the same displacement. The work-energy equation can be expressed in Equation (7), as shown in Fig. 5.

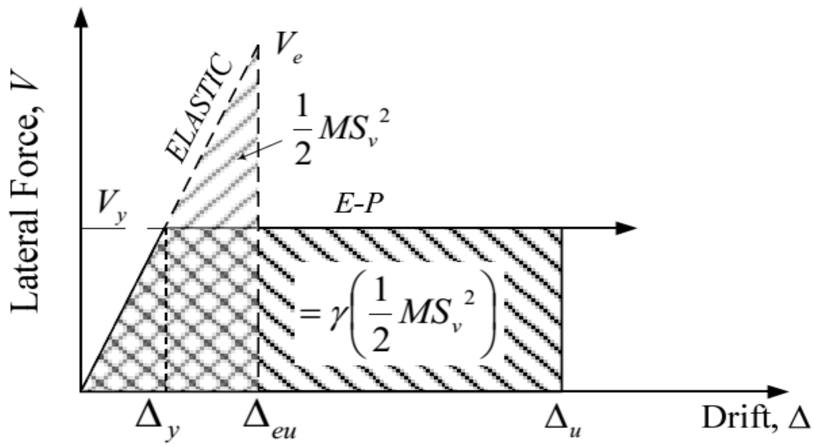


Fig. 5. Elasto-plastic curve of the structure[25]

$$(E_e + E_P) = \gamma\left(\frac{1}{2}MS_v^2\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}S_a g\right)^2 \tag{7}$$

Where E_e and E_P are respectively the elastic and plastic components of energy required to push the structure up to the target displacement, M is the mass of the structure, Sv is the pseudo-velocity spectrum of the design, Sa is the pseudo-acceleration spectrum of the design, and T is the main period of the structure. The energy correction coefficient depends on the structural coefficient of μs and the reduction coefficient $R\mu$ resulting from Equation 8.

$$\gamma = \frac{2\mu_s - 1}{R_\mu} \tag{8}$$

Studies on concrete structural systems show that due to the complexities and effects such as pinching in the concrete behavioral cycle, the mentioned relationships are not suitable for the concrete system, so Equation 7 is modified into Equation 9 by using the C2-factor method.

$$E_d = \frac{1}{2}\gamma^*.M.\left(\frac{T}{2\pi}S_a g\right)^2 = \frac{1}{2}\frac{2\frac{u_r/u_y}{C_2} - 1}{\left(R_\mu^*\right)^2}.M.\left(\frac{T}{2\pi}S_a g\right)^2 \tag{9}$$

The general trend of the energy capacity spectrum method is shown in figure 6:

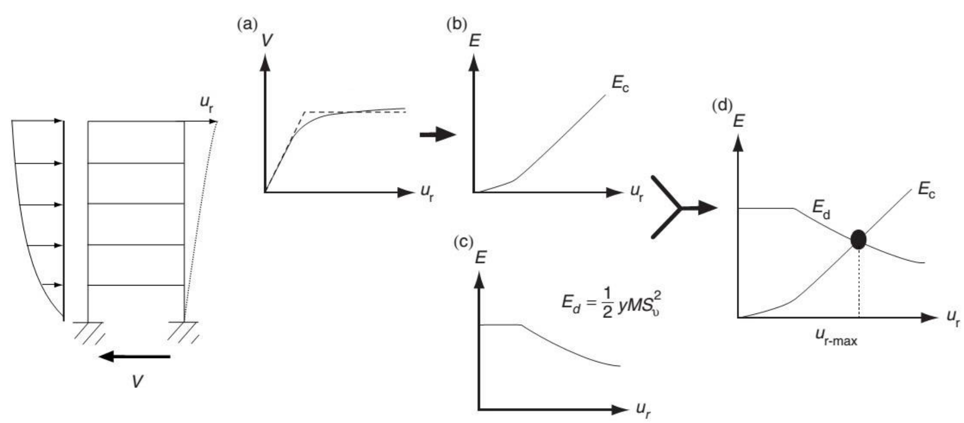


Fig. 6. (a) Pushover curve (b) Energy capacity spectrum (c) Energy demand spectrum (d) Performance point[25]

Figure 6 shows the general trend of the energy capacity spectrum method to estimate the structural performance point. The structural shear-based curve, against the roof displacement, obtained directly from the pushover analysis using a suitable lateral load pattern, is shown in section a, while the energy spectrum of the structure can be represented by the numerical integral of the force and the resulting displacement on the surface of each story, as shown in part b. The energy spectrum of the structural demand is also calculated with respect to the right side of Equation 9 (part c of Fig. 6), which is finally obtained from the intersection of the two spectra of capacity and demand according to section d of the structural performance point.

5. Introducing Structural Model and Software

The structural models used in this study relate to hospital structures with a special reinforced concrete moment frame system designed in accordance with the Iranian Seismic Code Standard 2800 [37], Fourth Edition with four 6-meter openings and a very high-risk area (A = 0.35 g) and built on Type 2 soil (350 m / s <Vs <750 m / s). A load of 27 kN / m was applied to the beams of the stories and details of the models' implementation are presented in Table 2.

To model the behavioral pattern of nonlinear concrete, the model of Mander *et al*. [34] which is a single-axis, nonlinear, confined and fixed model, was used. To determine the mechanical properties of concrete materials, capacity of compressive stress of cylindrical concrete specimen f_c, capacity of tensile stress of concrete specimen f_t, strain at maximum stress point E_c, and specific weight of concrete γ are presented in the software, as presented in Table 1 and shown in Figure 7.

Etabs2015 software was used for the initial design, the equivalent static analysis method was used for the 5-story structure and the spectral analysis method was used for the 10- and 15-story structures. The nonlinear model of frames was modeled in SeismoStruct2016 software and nonlinear static and dynamic analyses as well as modal analysis were performed in that software. MATLAB was also used to carry out the calculations required for idealizing pushover diagrams.

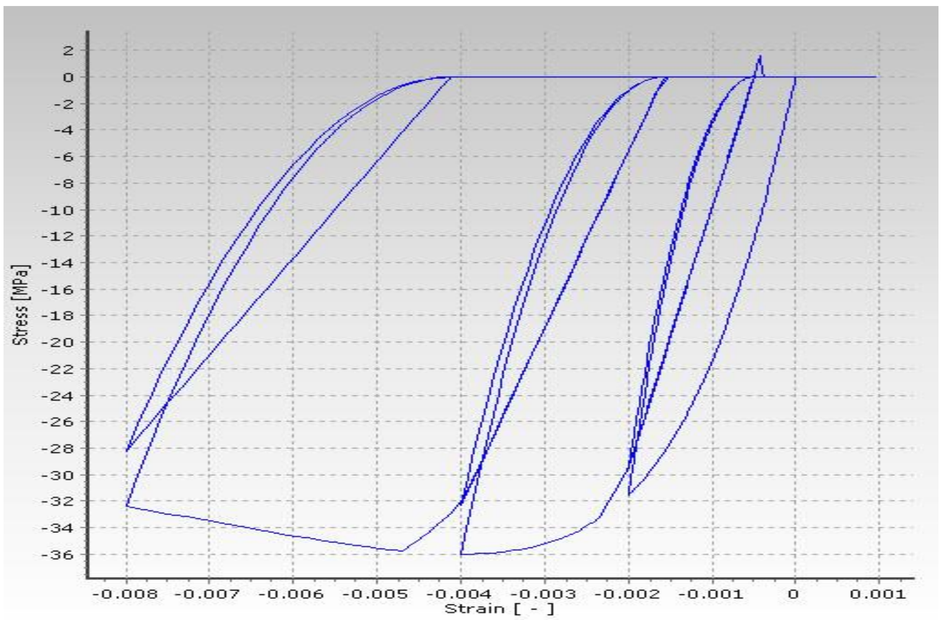


Fig. 7. Diagram of nonlinear behavior of concrete materials

Table 1. Mechanical properties of nonlinear concrete materials

f_c (MPa)	f_t (MPa)	E_c (GPa)	ε_c	γ (kN/m^3)
30	2.2	26.6	0.002	24

6. Improving the Energy Capacity Spectrum Method

As mentioned in the explanation of the energy capacity spectrum method, the structural energy capacity curve is obtained by applying the numerical integral of the force applied to each story over the displacement resulting from that force. Therefore, it is necessary at each step to analyze the force of each story and the displacement of its center of mass. Obviously, with this in mind, the computation volume and time required to determine the structural capacity curve increase with an increase in structural height.

Another important point in this case is the difficulty of using this method in the adaptive pushover analysis mode. As explained in Section 3, the load pattern in the adaptive pushover mode is changes at each step due to the variations in stiffness and high mode effects and the ratio of the power distribution between the stories does not remain constant, thus making it difficult to determine the force of each story at different steps for numerical integration.

In this study, an alternative method is proposed to achieve the capacity curve without the need for the story force at each step The capacity curve obtained from the different load patterns of this solution is then compared with the original method.

In the proposed method, instead of using the forces and displacements of all stories, the displacement of the roof is transformed using the relationships described below to the displacement of a single-degree system in accordance with the first mode of the structure's vibration.The numerical integral of the structural base shear against the displacement of the equivalent single-degree-of-freedom system serves as the energy capacity spectrum of the structure.

$$S_d = \frac{u_r}{\Gamma_1 \Phi_{N1}} \tag{10}$$

Where u_r is the displacement of the structural control point, S_d is the displacement of an equivalent degree-of-freedom system, Φ_{N1} is the value of the modal shape of the roof in the first mode and Γ_1 is the participation factor of the first mode of vibration obtained from Equation 11.

$$\Gamma_1 = \frac{\sum m_i \Phi_{i1}}{\sum m_i \Phi_{i1}^2} \tag{11}$$

In the above equation, m_i represents the mass of i^{th} story.

A comparison of the energy spectrum obtained from the improved method with the original energy capacity spectrum and the capacity spectrum obtained from the numerical integral of the roof displacement and the total base shear of the total structure for a 5-story system with a load pattern according to the first mode of vibration is shown in Fig. 8.

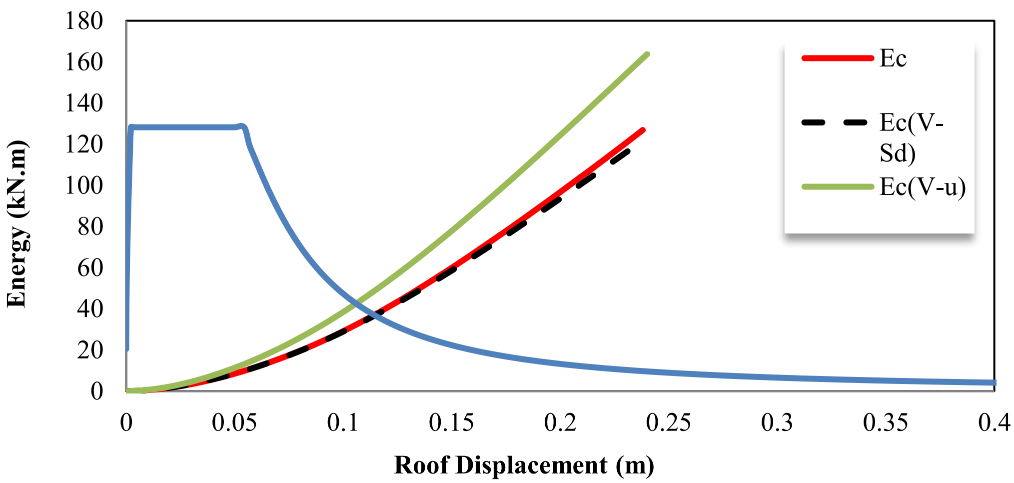


Fig. 8. Comparison of the energy capacity spectrum obtained by different methods - first vibration mode load pattern, 5-story frame

Fig. 8 shows the graph of the energy capacity curve obtained from the numerical integral of the force and its resultant displacement in each story, the Ec (Vu) energy capacity curve obtained from the numerical integral of the base shear and the displacement of the roof of the multi- degree-of-freedom system and Ec (V-Sd) the energy capacity curve (proposed in this study) ,obtained from the numerical integral of the base shear and the displacement of the equivalent single-degree-of-freedom system. It shows that the proposed method in this study for determining the energy capacity curve of Ec (V-Sd) aligns well whit the capacity curve of the main energy of Ec, especially at the confluence point with the Ed demand curve.

This study was conducted for 5- and 10- story concrete structures with different lateral load patterns that yielding acceptable results as shown in Fig. 8.

7. Selected Earthquake Records

In this study, we used the proposed 7 FEMA-P695 records and 18 FEMA-440 records that match the wave velocity in the soil of the record-keeping seismic station with the selected site soil, and the records were coordinated with the elastic design spectrum of the site as, presented in Table 3.

Table 2. Selected earthquake records

#	Earthquake	Year	Station	Moment Magnitude (Mw)	Distance (km)	Vs30 (m/sec)	PGA(g)
			Proposed Earthquake FEMA440				
1	"Morgan Hill"	1984	"Gilroy - Gavilan Coll."	6.19	14.8	730	0.12
2	"Morgan Hill"	1984	"Gilroy Array #6"	6.19	9.9	663	0.29
3	"Loma Prieta"	1989	"APEEL 7 - Pulgas"	6.93	41.9	415	0.16
4	"Loma Prieta"	1989	"Anderson Dam (Downstream)"	6.93	20.3	489	0.25
5	"Loma Prieta"	1989	"Berkeley LBL"	6.93	79.3	624	0.12
6	"Loma Prieta"	1989	"Gilroy - Gavilan Coll."	6.93	10	730	0.36
7	"Loma Prieta"	1989	"Gilroy Array #6"	6.93	18.3	663	0.17
8	"Loma Prieta"	1989	"Hollister - SAGO Vault"	6.93	30.2	621	0.06
9	"Loma Prieta"	1989	"Monterey City Hall"	6.93	44.4	639	0.07
10	"Loma Prieta"	1989	"SF - Diamond Heights"	6.93	71.3	583	0.11
11	"Loma Prieta"	1989	"Yerba Buena Island"	6.93	75.2	660	0.07
12	"Landers"	1992	"Silent Valley - Poppet Flat"	7.28	50.9	659	0.05
13	"Landers"	1992	"Twentynine Palms"	7.28	41.4	635	0.08
14	"Northridge-01"	1994	"Antelope Buttes"	6.69	46.9	573	0.07
15	"Northridge-01"	1994	"Castaic - Old Ridge Route"	6.69	20.7	450	0.51
16	"Northridge-01"	1994	"Lake Hughes #1"	6.69	35.8	425	0.09
17	"Northridge-01"	1994	"Littlerock - Brainard Can"	6.69	46.6	486	0.07
18	"Northridge-01"	1994	"Mt Wilson - CIT Seis Sta"	6.69	35.9	680	0.13
			Proposed Earthquake FEMA-P695				
19	"Northridge-01"	1994	"Beverly Hills - 14145 Mulhol"	6.69	17.2	356	0.52
20	"Kobe_ Japan"	1995	"Nishi-Akashi"	6.9	7.1	609	0.51
21	"Kocaeli_ Turkey"	1999	"Arcelik"	7.51	13.5	523	0.22
22	"Chi-Chi_ Taiwan"	1999	"TCU045"	7.62	26	705	0.51
23	"Friuli_ Italy-01"	1976	"Tolmezzo"	6.5	15.8	505	0.35
24	"Manjil_ Iran"	1990	"Abbar"	7.37	12.6	724	0.51
25	"Hector Mine"	1999	"Hector"	7.13	11.7	726	0.34

8. Results and Discussion

8.1 Lateral Load Distribution Patterns

One of the conventional methods for evaluating the accuracy of load distribution patterns is to compare the relative displacement of the stories at the point when the structural control point reaches the target displacement with the mean relative displacement of the stories obtained from time history analysis for different records. The disadvantage of this approach is that the relative displacements of the structures are compared with the results of the dynamic analysis only at one point, and the displacements are not considered at different functional levels.

This study used an alternative method that attempts to compare the displacement performance of the structure under the lateral load distribution pattern at different levels with the results of time history analyses. Thus, for each displacement value, the maximum control point in time history analysis under each record is compared with the corresponding value in the diagram, and the degree of deviation between the relative displacements of the stories from the time history analysis is calculated using Equation (12).

$$Bias = e^{\frac{\sum_1^n \ln\left(\frac{\theta'}{\theta}\right)}{n}} \tag{12}$$

Where n is the number of earthquake records, θ is the maximum relative displacement ratio in the stories derived from the time history analysis, and θ′ is the relative displacement ratio in the stories derived from the pushover analysis.

An example of this comparison for a 10-story frame relating to 4 earthquake records is presented in Figure 9.

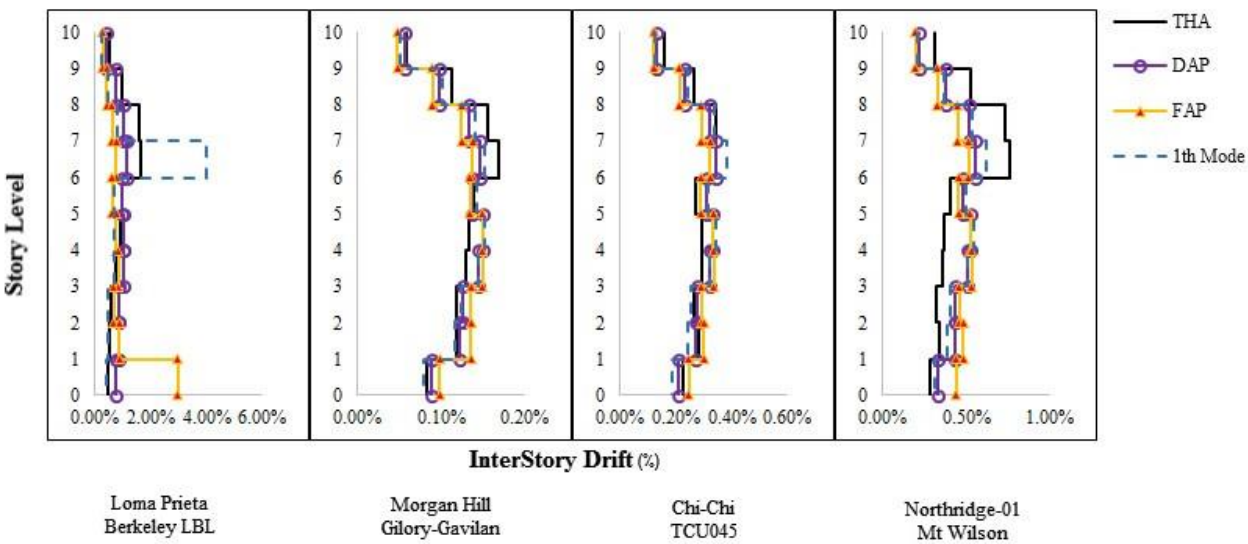


Fig. 9. Comparison of the relative displacement of stories in roof displacement values corresponding to time history analysis under different records

Fig. 9 compares the relative displacement values obtained from time history analysis (THA) under the four records with the relative displacement of the stories from pushover analysis using force-based adaptive patterns (FAP), displacement adaptive patterns (DAP), and lateral load pattern according to the first vibration mode at the corresponding roof displacement.

The comparison was made for each of the 25 records presented in Table 3 and the relative deviations of the relative displacements of each story were calculated using Equation 12.

Figure 10 presents a comparison of the deviations of different lateral load distribution patterns for a regular 5-story frame

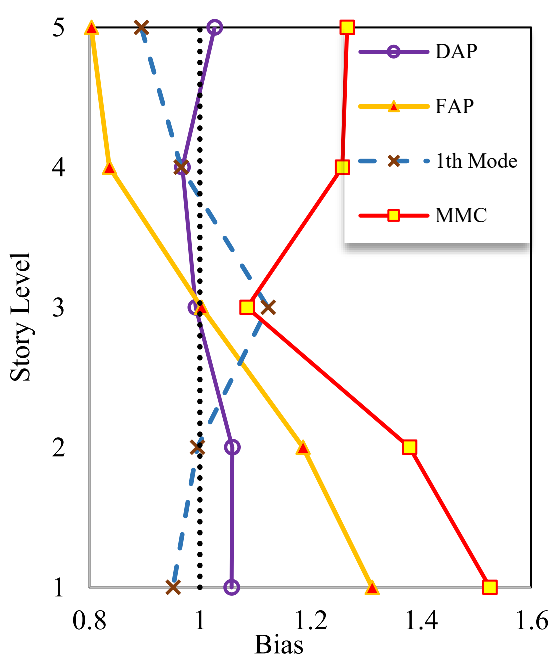


Fig. 10. Comparison of the deviation of different patterns of lateral load distribution across various stories - regular 5-story frame

In Fig. 10, deviations greater than 1 indicate conservative overestimates, while values less than 1 indicate underestimates. Obviously, the closer the calculated values are to 1, the more accurate the lateral load distribution pattern is.

Fig. 10 shows that the FAP-simulated load distribution pattern yielded overestimates in the lower stories of both the upper and lower frames. The MMC modal load distribution also yielded extremely high estimates across all stories. Between the load pattern according to the first mode of vibration and the displacement adaptive pattern, the latter yielded more accurate results. The coefficient of variation was used for the quantitative comparison of the accuracy of different load distributions, as derived from Equation (13)

$$CoV = \frac{\sigma}{|\mu|} \tag{13}$$

Where σ is the standard deviation and μ is the mean value the relative displacement deviation of the stories. Obviously, the lower the variation level of the data for a lateral load distribution pattern, the greater the accuracy of the load distribution.

Table 4 presents a quantitative comparison between the accuracy of the distribution of different loads for a regular 5-story frame. Since Fig. 10 shows the overestimated values of the modal load distribution estimates, we have avoided providing quantitative values of the deviation of this method.

Table 3. Quantitative comparison of the accuracy of different lateral load pattern

Story	DAP	FAP	1th Mode
	Bias		
5	1.026	0.803	0.893
4	0.968	0.835	0.965
3	0.992	1.003	1.123
2	1.058	1.186	0.995
1	1.057	1.312	0.951
Average	1.02	1.028	0.985
deviation	0.035	0.196	0.076
COV	0.034	0.191	0.077

The outputs of Table 4 show the accuracy of the displacement adaptive pattern compared to the other two patterns.

This comparison was also made for irregular 5-story frames, as well as regular and irregular 10- and 15- story frames, which are presented in the following diagrams.

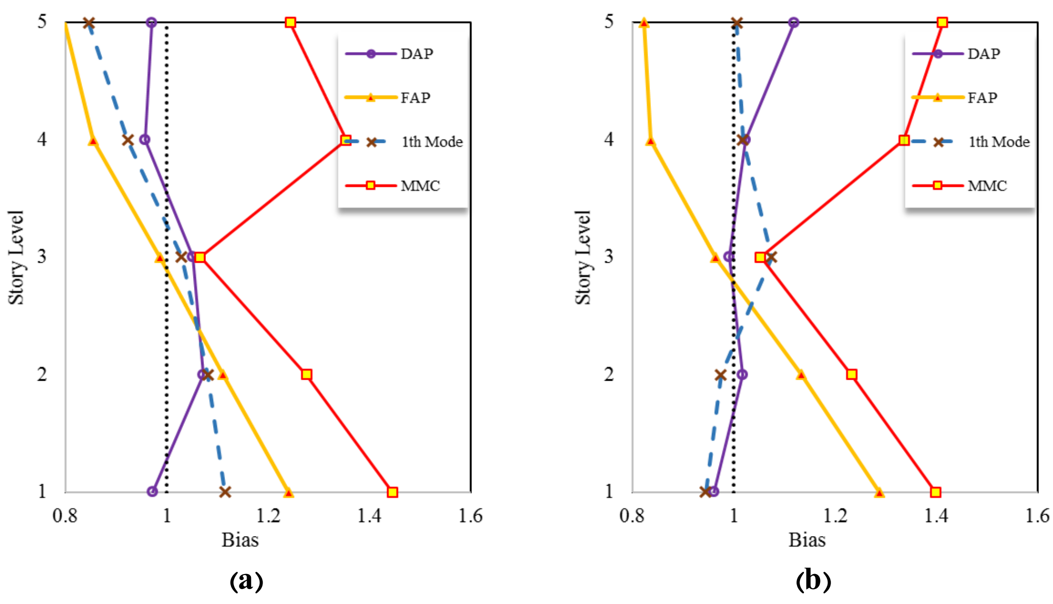
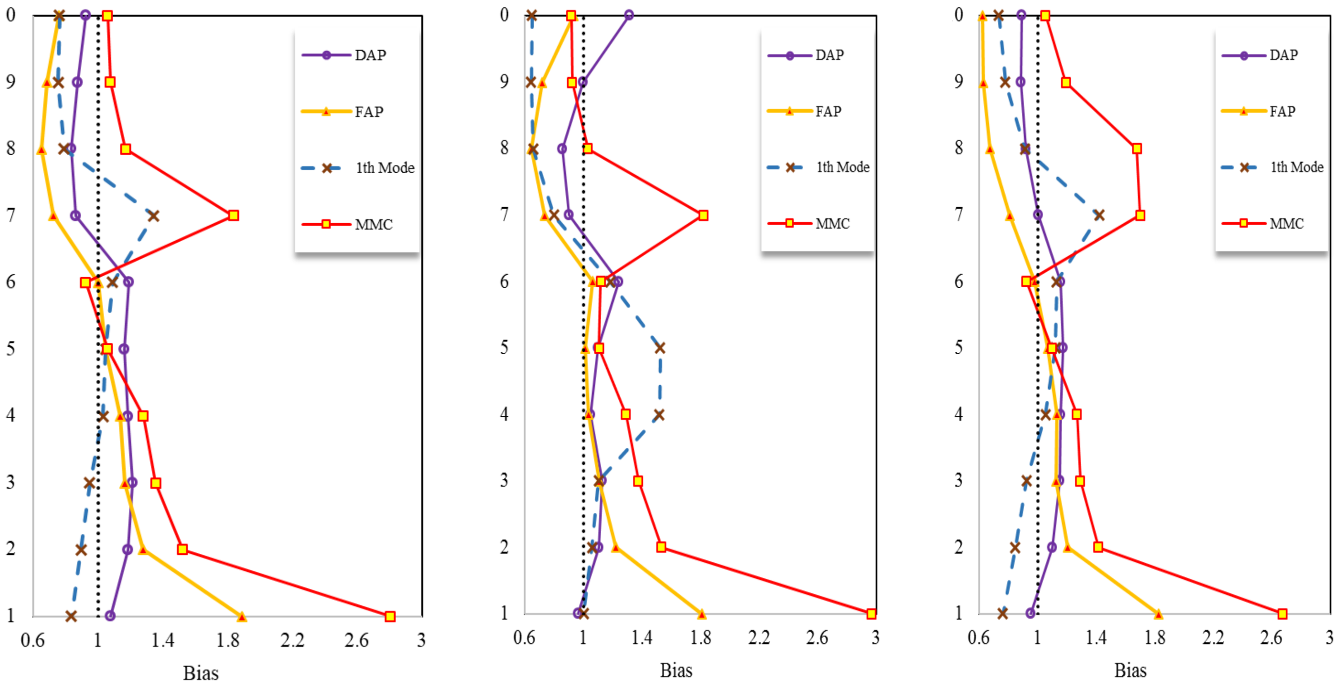
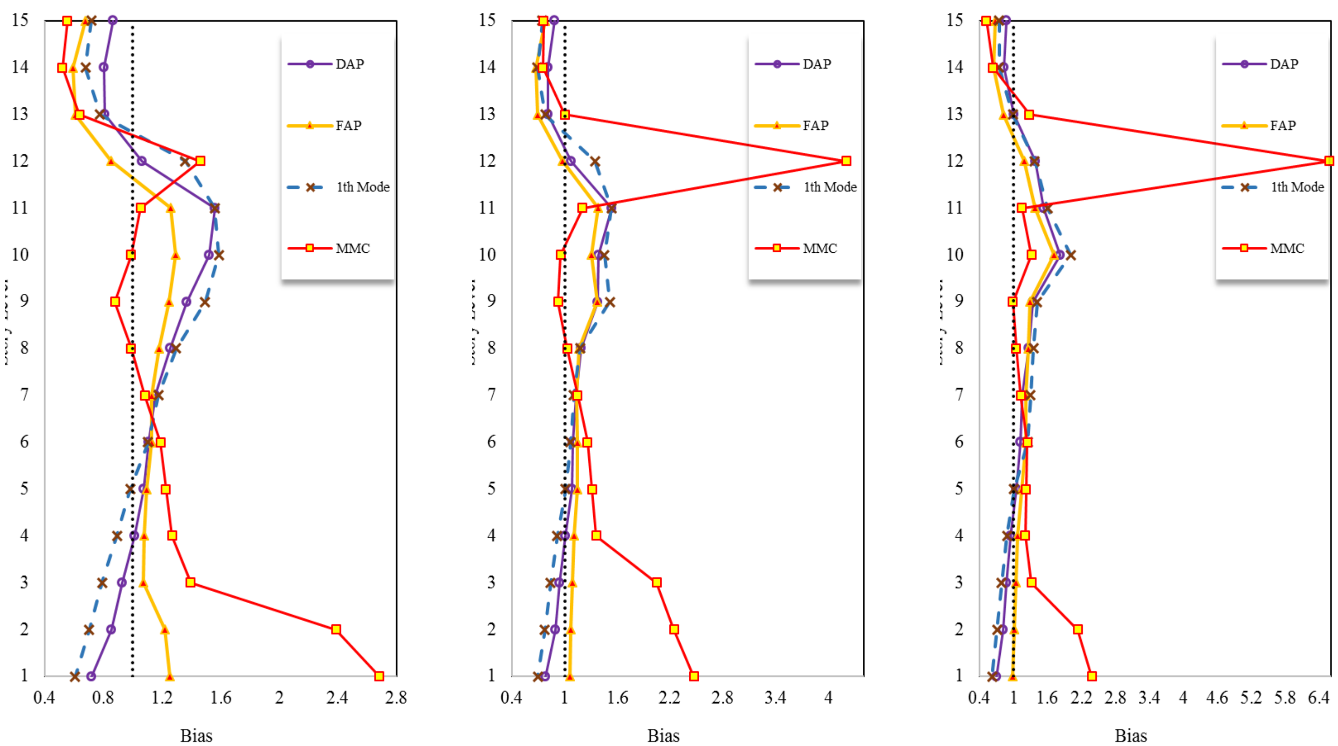


Fig. 11. Comparison of the deviation of different lateral load patterns in different stories (a) Irregular 5-story frame in the middle story (b) Irregular 5-story frame in the final story



(a) **(b)** **(c)**

Fig. 12. Comparison of the deviation of different patterns of lateral load distribution across different stories a. Regular 10-story frame, b. Irregular 10-story frame in the middle story, c. Irregular 10-story frame in the final story



(a) **(b)** **(c)**

Fig. 13. Comparison of the deviation of different patterns of lateral load distribution across different stories a. Regular 15-story frame, b. Irregular 15-story frame in the middle story, c. Irregular 15-story frame in the final story

The results of the quantitative comparison between different load distribution patterns expressed as coefficients of variation are presented in Table 5.

Table 5. Comparison of coefficient of variation of different lateral load patterns

Frame	Coefficient of variation		
	DAP	FAP	1th Mode
Irregular 5-story frame in middle story	0.047	0.163	0.1
Irregular 5-story frame in final story	0.052	0.177	0.044
Regular 10-story frame	0.149	0.342	0.184
Irregular 10-story frame in middle story	0.134	0.323	0.328
Irregular 10-story frame in final story	0.116	0.359	0.221
Regular 15-story frame	0.234	0.223	0.316
Irregular 15-story frame in middle story	0.206	0.195	0.279
Irregular 15-story frame in final story	0.267	0.235	0.338

The outputs of Tables 4 and 5, as well as Figures 10 to 13, show that the displacement adaptive pushover (DAP) distribution is, in most cases, more accurate than other load distribution patterns and can be used as a suitable lateral distribution pattern for evaluating the different performance point determination methods introduced earlier.

8.2 Determining the Performance Point

As mentioned in the previous section, among the different lateral load distribution patterns, the DAP model exhibited better accuracy than the others. The performance points of the frames were determined using the four described methods based on the resultant pushover curve. The relative displacements of the frame stories at the time of reaching the performance point were calculated separately for each method and compared with the results of the maximum dynamic time history analysis. Fig. 14 shows a comparison of the relative displacement of the stories at the moment of reaching the target displacement using the four aforementioned capacity-based methods alongside the time history analysis results for the regular and irregular 5-story frames.

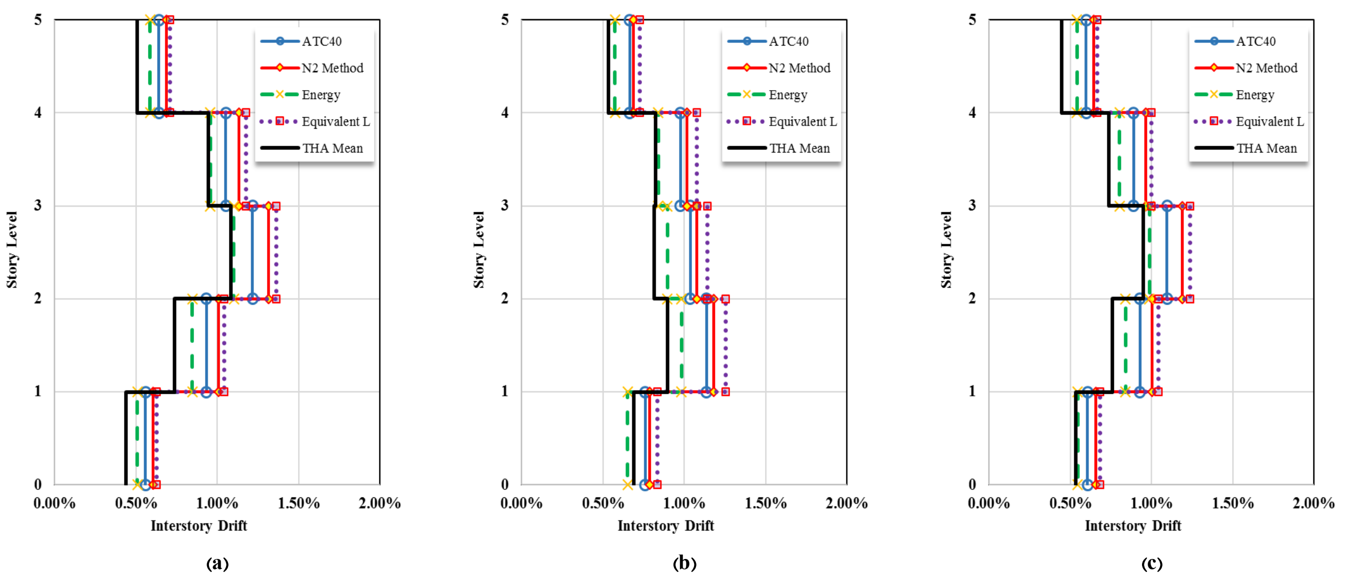
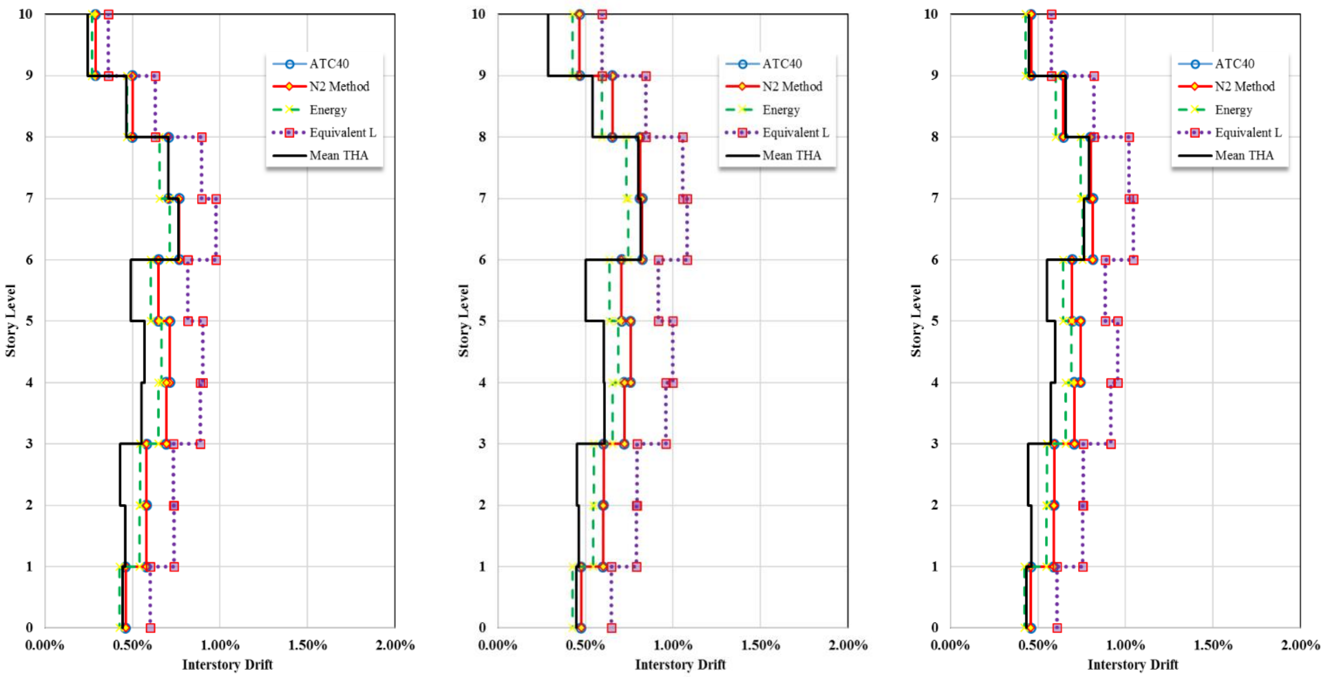


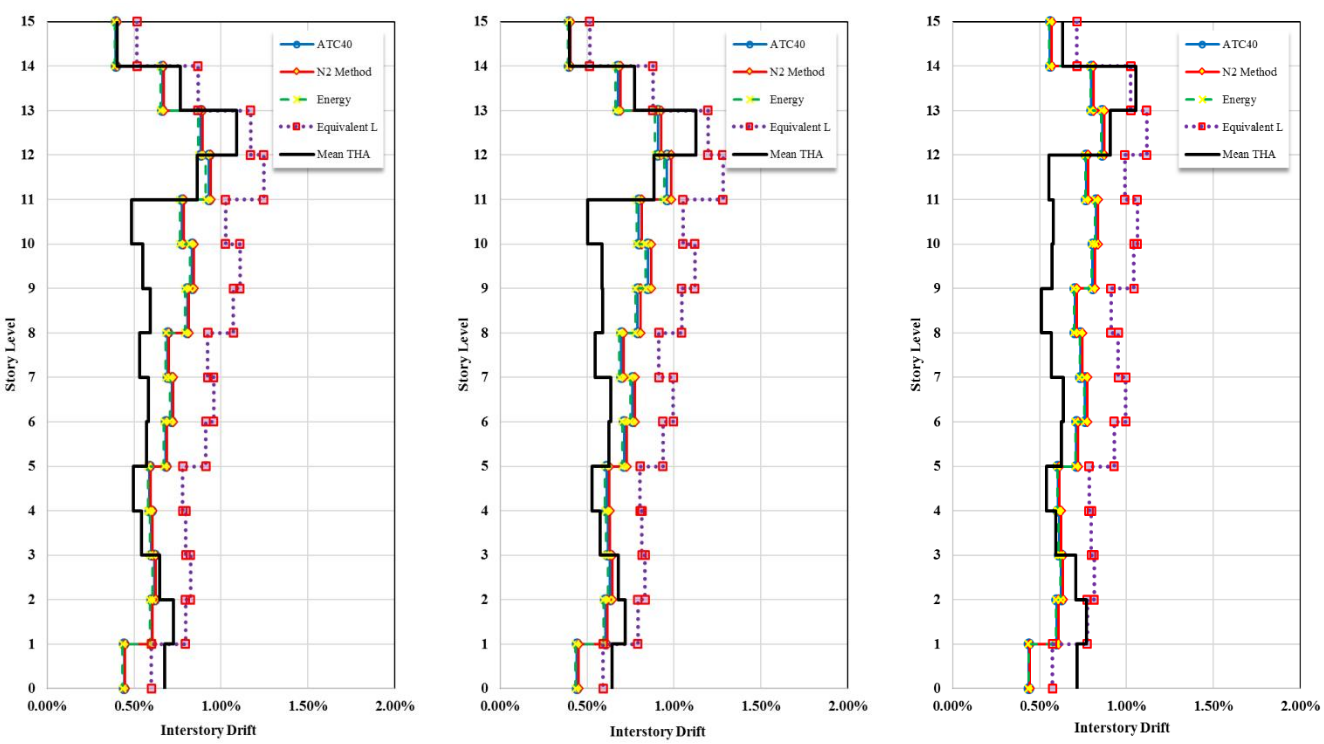
Fig. 14. Comparison of displacement results from different methods of performance point determination with time history analysis output a. Regular 5-story frame, b. Irregular 5-story frame in the middle story, c. Irregular 5-story frame in the final story

Fig. 14 shows the remarkable closeness of displacement responses to the results of nonlinear time history analysis when using the energy spectrum method. This comparison was also made for regular and irregular 10- and 15- story frames, as presented in Figures 15 and 16.



(a) (b) (c)

Fig. 15. Comparison of displacement results from different methods of performance point determination with time history analysis output a. Regular 10-story frame, b. Irregular 10-story frame in the middle story, c. Irregular 10-story frame in the final story



(a) (b) (c)

Fig. 16. Comparison of displacement results from different methods of performance point determination with time history analysis output a. Regular 15-story frame, b. Irregular 15-story frame in the middle story, c. Irregular 15-story frame in the final story

To perform a quantitative comparison between different performance point determination methods, the relative displacement error values calculated for each story, compared with time history analysis, were obtained using Equation 14 for each of the performance point determination methods and compared in Table 6.

$$Eror(\%) = \frac{|\Delta_{THA} - \Delta_{NSA}|}{\Delta_{THA}} \tag{14}$$

Where Δ_{THA} is the relative displacement of the story derived from dynamic time history analysis, and Δ_{NSA} is the relative displacement of the story resulting from nonlinear static analysis.

In Table 6, the error values were calculated for each story, and the mean error values of the stories were obtained with the following results.

Table 6. Mean error in estimating the displacement needed for different methods of performance point determination

Frame	ATC-Error	N2-Eror	Equivalent L-Error	Energy-Error
Regular 5-story frame	0.209	0.303	0.349	0.1
Irregular 5-story frame in middle story	0.215	0.26	0.336	0.07
Irregular 5-story frame in final story	0.211	0.311	0.336	0.093
Regular 10-story frame	0.175	0.197	0.49	0.13
Irregular 10-story frame in middle story	0.242	0.263	0.626	0.169
Irregular 10-story frame in final story	0.154	0.138	0.474	0.112
Regular 15-story frame	0.232	0.236	0.492	0.224
Irregular 15-story frame in middle story	0.211	0.222	0.451	0.204
Irregular 15-story frame in final story	0.234	0.242	0.434	0.232

The results of Figures 14 to 16 and Table 6 show that the energy capacity spectrum method is more accurate in estimating the structural performance point than other capacity spectrum-based methods.

9. Conclusion

The use of nonlinear static (pushover) analysis as an efficient analytical method for estimating the seismic performance of structures is simpler and faster than d ynamic time history analysis, which is why it has been a common subject in many earthquakes engineering studies. Pushover analysis is a simple option for estimati ng the structural capacity of a structure in the plastic range and can be used to identify weak structural areas. However, weaknesses such as disregarding the effects of higher modes, changes in stiffness and member resistance during the analysis, as well as ignoring earthquake recurrence effects, have led to many efforts to impr ove this analysis, increase its accuracy, and address the mentioned weaknesses through innovative methods. This study first evaluated the accuracy of three types of lateral load distribution patterns (traditional, modal, and adaptive) in comparison with dynamic time history analysis and compared the deviation values.

Then, four methods of determining the point of performance based on the capacity spectrum are introduced and the error rate of each in estimating the structura l displacement needs is compared with the time history analysis.

A comparison of the outputs of different methods shows that:

The displacement adaptive pattern (DAP) in each of the 9 regular and irregular frames has, on average, less deviation than the other load patterns, indicating an appropriate estimation of the structure's displacement using this lateral load pattern at all functional levels.

- The modal lateral load distribution model resulting from the algebraic sum of the MMC loads in all frames yielded high (and in some cases very hig h) estimates, indicating the impracticality of using this load pattern given its multi-stage analysis.
- The energy capacity spectrum method for estimating the structural performance point yielded better results in all modes of frames examined, includi ng 5-story (low height) frames, 10- story (medium-height) frames and 15-story (relatively high-height) frames, as well as in regular and irregular co nditions (in the middle and final stories) than the three proposed methods of ATC-40, N2 and equivalent linearization methods.
- The energy spectrum method has high precision as well as a simpler and faster process than the other three methods. Moreover, it has the advantage of being non-repetitive in the execution process.

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