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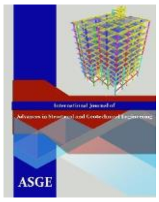

Evaluation of the Egyptian code for Seismic design of regular frames using the performance-based design methods

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Abstract

In the present decades, more advanced seismic nonlinear analysis has become possible. ATC-40, FEMA-356, Eurocode, the N2 method, and FEMA-440 are popular performance-based seismic design (PBSD) procedures. These procedures determine the performance level of the structure and the target displacement using the nonlinear static behavior of the structure and the damped response earthquake spectrum. The main objective of this paper is to evaluate the performance of regular frames designed by the Egyptian code equivalent static load method using several performance design methods. The studied regular RC frames are six, ten, and fourteen-story limited ductile concrete frames. The SeismoStruct nonlinear finite element program was used to perform the pushover nonlinear static analysis. The maximum inelastic displacement and the performance level of the reinforced concrete buildings were determined for each performance method. In general, it was concluded that the equivalent static method is conservative, especially for tall frames, because it is based on simple linear assumptions. The results of this paper showed that the performance level of regular concrete frames designed by the ECP-201 seismic code method subjected to 0.15g and 0.3g, was a life safety level. Also, the capacity spectrum method (ATC-40) gave the lowest target displacement in most cases, and the displacement coefficient method (FEMA-440) gave the highest value.

1. Introduction

Earthquakes cause severe damage to property and life. Modern design procedures known as "performance-based seismic design" (PBSD) are used to predict how a structure will behave when subjected to different seismic excitations. PBSD procedures have been suggested to overcome the drawbacks of the Force-Based Seismic Design (FBSD) methods. The FBSD procedures determine the structural period based on cracked stiffness and distribute the seismic forces among elements, ignoring that structural elements can be forced to yield simultaneously. It also relies on the force reduction factor value (R), which is an essential seismic design tool that defines the level of ductility expected in structural systems during major earthquakes. Moreover, it is used to reduce the design forces in earthquake-resistant designs and accounts for damping, energy dissipation capacity, and over-strength of the structure. The FBSD procedures include displacement criteria as a final check after detailing the structure by checking inter-story drift requirements and comparing them to code-specified displacement limits. A displacement amplification factor is used to convert the displacement resulting from the elastic analysis to its inelastic displacement. However, the Equivalent Static Load Method is still the most widely used in all seismic codes and standards, due to its efficiency and simplicity[1]. In this method, the earthquake inertial forces are represented by equivalent static lateral loads distributed all over the building height. This simple design method is restricted to regular buildings of relatively low rise.

Relatively new nonlinear Methods called the performance-based seismic (PBSD) methods have a

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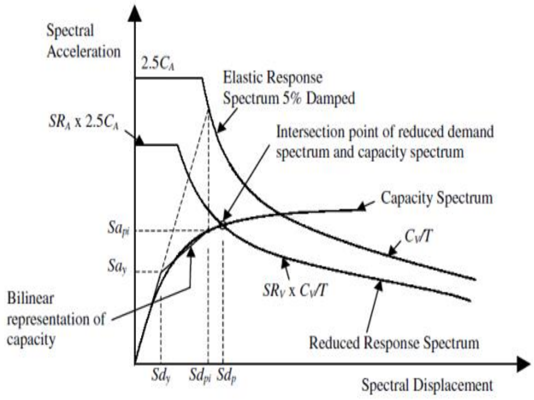
common basic principle which is combining in the nonlinear static analysis, referred to as pushover analysis (POA). In POA, the multi-degree-of-freedom (MDOF) structure is subjected to incremental increasing lateral loads until reaching failure or a predefined target maximum displacement. The PBSD methods implemented in this paper are the Capacity Spectrum Method (ATC-40) [2], the N2 Method [3],and the Improved Procedures for Displacement Modification (FEMA-440) [4]. Pushover nonlinear static analysis would be used as a reliable and effective tool for analyzing and obtaining the performance levels of the structure. One of the advantages of performance-based analysis is the ability to estimate seismic demand and capacity with a reasonable degree of accuracy. The predicted seismic performance can be assessed according to the performance categories defined by the same procedure. This can ensure the safety and stability of structures subjected to different seismic levels [5].

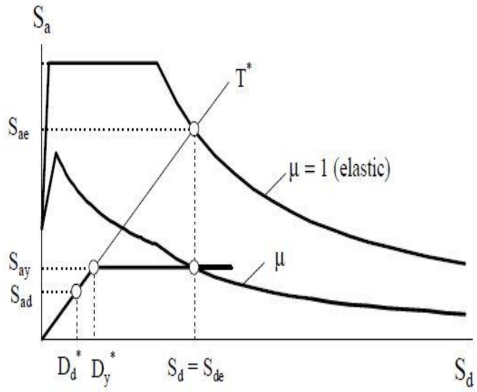
The capacity spectrum method (ATC-40) [2] is a nonlinear static analysis method for estimating displacements and comparing the capacity of a structure according to the demands of earthquake ground motion. The inelastic strength and displacement spectra are used for the determination of an earthquake demand. This method recognizes that when the structure is shaken beyond its yield point, the effective damping due to the hysteretic behavior is included and the reduced response spectrum is obtained. Both the capacity curve and reduced demand response are converted into acceleration–displacement response spectrum (ADRS) format. The maximum structural response is the point where the structural capacity curve crosses the reduced demand spectrum. The disadvantage of this method is that the capacity spectrum method usually requires a lot of iterations to find the exact intersection point, and therefore this procedure is more appropriate for evaluation and retrofit purposes than for designing new structures [5].

Figure 1(a) illustrates the capacity spectrum procedure and the performance point finding. A bilinear representation of the capacity spectrum is needed to estimate the effective damping and appropriate reduction in spectral demand. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated point, then that point is the performance point. After determining the performance point for a single degree of freedom (SDOF) it will convert to a multi-degree of freedom (MDOF).

The N2 Method is a relatively simple nonlinear method for the seismic analysis of structures proposed by Fajfar [3], and it has been implemented by the EC8 code [6]. The capacity curve is produced from the pushover analysis, while the inelastic demand spectra are obtained by reducing the elastic spectra using reduction factors mainly based on the calculated structural ductility. The acceleration-displacement (AD) inelastic spectra are constructed, and the demand quantities can be determined as shown in Fig.1 (b). Generally, the results of the N2 method are reasonably accurate, provided that the structure oscillates predominantly in the first mode. The disadvantage of this method is that it has unacceptable results for soft soil and higher buildings because it does not consider the effectiveness of higher modes [5].



(a) ATC -40



(b) N2 method

Fig. 1:(a) Capacity Spectrum method to Determine Performance Points [2] and (b) Elastic and inelastic demand spectra versus capacity diagram in the N2 method [3]

Displacement Coefficient method FEMA-440 [4] is a simple PBSD method for estimating the target displacement δ_t. The target displacement refers to the displacement of the characteristic node on the roof of a structure, during a seismic event. It does not require converting the capacity curve into the corresponding spectral coordinates. The target displacement is an estimate of the maximum inelastic deformation demands of the studied building, which is defined by Eq.1,

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^{\ 2}}{4\pi^2} g \qquad (1)$$

where:
C_0: A modification factor used to relate the spectral displacement of an equivalent SDOF system to the roof displacement of the structure MDOF system.
C_1: A modification factor to relate expected maximum inelastic displacements to elastic displacements.
C_2: A modification factor to represent the effect of stiffness degradation and strength deterioration on maximum displacement response.
After determining the performance point for MDOF, the inter-story drift ratio is determined, and the performance

level of the structure is defined according to the procedure limits defined by FEMA-440 [4] as given in Table 1.

Table 1: Structural performance levels according to FEMA-440[4].

Elements	Type	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
Concrete Frames	Inter-story Drift	1%	2%	4%

Several recent studies participated in the development of PBSD methodologies, applying different methods and according to different codes. Some of the research employed FEMA-356, ASCE-41, FEMA-440, and ATC-40 to determine the target displacement [5, 7, 8] [9, 10] [11, 12]. while others determined the performance level of frames [9]. Kasimzade et al.[13] attempted to raise the structure's maximum load-carrying capacity by adding a new criterion that required many plastic hinges in the structure, and the results showed that the load-bearing capacity of the frame-bearing system increased by 50% compared to that resulted from the code methods. On the other hand, some research evaluated seismic code design methods. Hakim et al. [14] evaluated the performance of buildings designed according to the Saudi Building Code (SBC) by ATC-40, FEMA-356, and FEMA-440. The research shows that SBC-designed buildings generally met the acceptance criteria for these methods. The ATC-40 gave the lowest target displacement, δt. However, all three methods indicated that the margin of safety against collapse is high and there are sufficient reserves of both strength and drift. Zameeruddin and Sangle [15] conducted a study to evaluate the performance using nonlinear static procedures, following the guidelines of Indian seismic codes. The obtained results showed a disagreement with the Indian seismic code provisions, especially towards the fundamental period, upper and lower bound values of the base shear-drift ratio, and the modification factor. The values of R obtained at IO, LS, and CP performance levels showed that IS 1893 overestimates the R factor. Mazumder and Ansary [16] obtained the target displacement of frames designed according to the Bangladesh National Building Code (BNBC 1993) for a moderate seismicity region with a peak ground acceleration of 0.15g. They stated that damage would be limited in this building as the yielding occurred up to the life safety limit. The performance of the structure can be described as good, as the building had a reasonable reserve strength under the design for earthquake ground motion. Mwafy and Elnashai [17] studied different structural systems subjected to different design peak accelerations (0.15g and 0.30g), assuming different design ductility levels (low, medium, and high). The results showed that pushover analysis was more appropriate for low-rise and short-period structures and that the triangular loading distribution was adequate to predict the response of such structures.

The objectives of this paper are:
1-Checking the accuracy of the current force-based seismic code design method.
2- Determination of target displacement and base shear for all frames.
3- Evaluation of the performance level of the regular frames designed according to the equivalent static method by the seismic Egyptian code, ECP-201[18].

2. Numerical study

In order to achieve the aforementioned objectives, regular frames are designed according to ECP-201[18] and ECP-203 [19]. For applying the performance-based design methods, SeismoStruct [21] has been used to conduct the nonlinear static analysis of RC frames. This program is a general-purpose finite-element package for nonlinear analysis of two and three-dimensional reinforced concrete under static and dynamic loading. The effects of geometric nonlinearities, material inelasticity, reinforcement yielding, and cracking are included in the analysis in this paper. Inelastic displacement-based frame elements were used to model columns and beams. In this study, it was assumed no shear collapse would occur in any structural member. Components of the element are shown in Fig.2 Pushover analysis is a nonlinear, static with gradually increasing predefined lateral loads with an inverted triangular shape increasing pattern along the building's height similar to that of the code lateral seismic force distribution [18]. The building's failure modes and plastic hinges are formed with the increasing lateral loading till failure or reaching a predefined drift value.

Description of the mathematical model and loading

The buildings selected for this study are six regular RC-framed structures with 6, 10, and 14 stories. All the selected buildings have a similar plan arrangement with three bays (6.0 m) in each direction, and the story height is 3.0 m for all stories, as shown in Fig. 3. Limited ductile frames were designed according to the guidelines of ECP-203 [19] and ECP-201[18] using the equivalent static method and were analyzed using the ETABS analysis program. The building frames were situated in seismic zones with 0.15g and 0.30g peak ground acceleration. Preliminary design considerations are tabulated in Tables 2 and 3. For the selected moment-resisting frame seismic design parameters, the value of the force reduction factor, R is 5, and the importance factor is 1. The floor system is a solid slab where the live load is 5.00 kN/m^2 and partitions, finishes, and slab self-weight are both assumed to be 16 kN/m^2. All the buildings are assumed to be founded on soil class C. The cross-section capacities have been computed by considering a characteristic cube concrete strength of 30 N/mm^2 and a yield strength of 360 N/mm^2 for both longitudinal and transverse steel.

3. Results and Discussions

Figures 4 and 5 show the pushover base shear-drift curves with the performance point for all buildings designed for peak ground accelerations of 0.15g and 0.3g, respectively. The curves represent the global behavior of the frame in terms of stiffness and ductility.

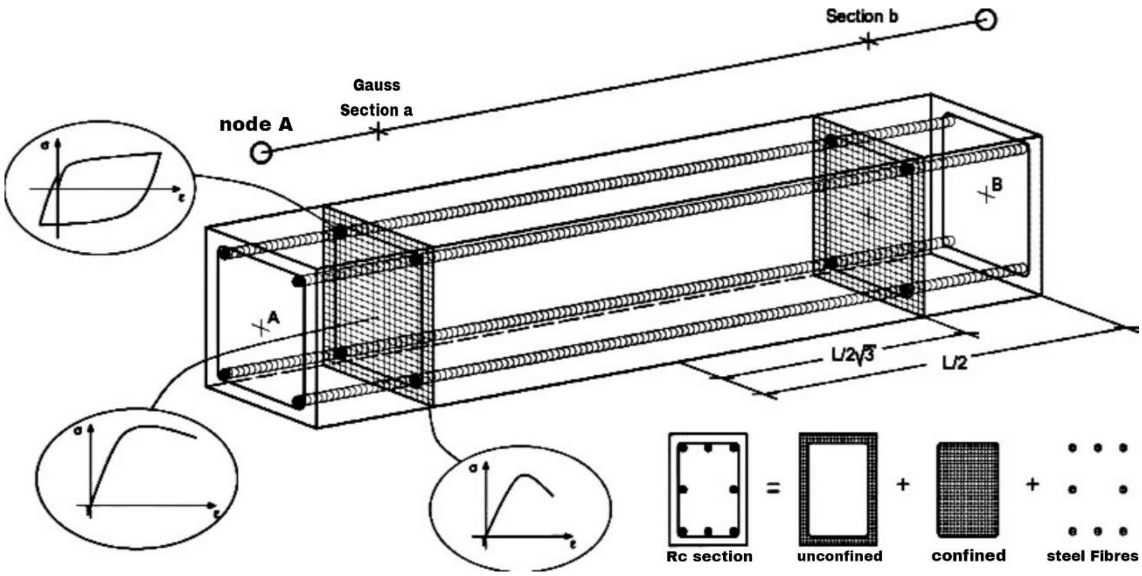
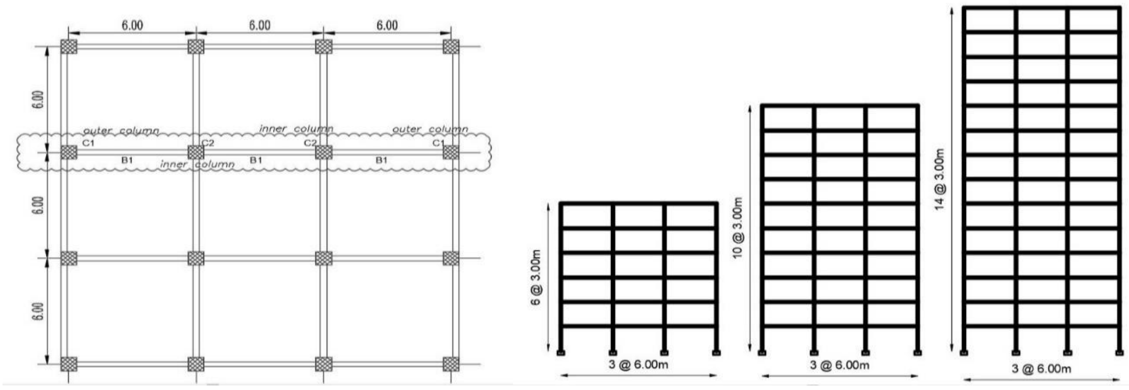


Fig. 2: Components of the inelastic displacement-based frame element [21].



(a) Plan. **(b) Elevations of the analyzed frames**

Fig. 3: Plan and elevations of studied RC buildings with moment resisting frames.

Table2: Member dimensions and reinforcement of the frames for ag=0.15g

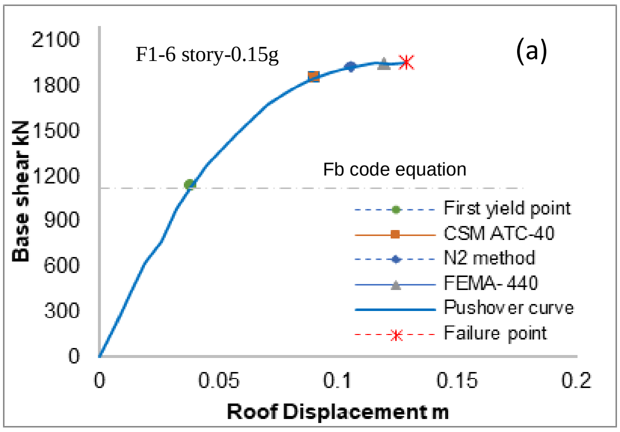
Building		Outer column		Inner column		Beams				
	Label	Size, mm	RFT	Size, mm	RFT	Size, mm	At support		At mid-span	
							Bottom	Top	Bottom	Top
6- story	F1	600*600	20T16	800*800	24T18	250*800	7T16	12T18	7T16	2T18
10-story	F2	700*700	20T18	950*950	36T18	250*850	8T16	12T18	8T16	3T18
14-story	F3	900*900	24T22	1250*1250	44T22	250*900	9T18	13T18	9T18	3T18

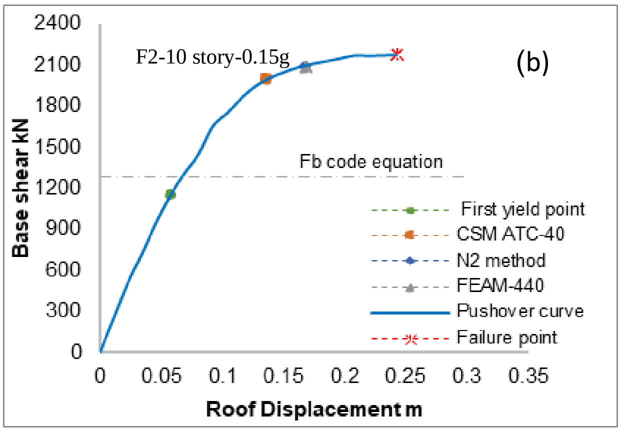
where, T= (High tensile steel symbol).

Table3: Member dimensions and reinforcement of the frames for ag=0.30g

Building		Outer column		Inner column		Beams				
	Label	Size, mm	RFT	Size, mm	RFT	Size	At support		At mid-span	
							Bottom	Top	Bottom	Top
6- story	F4	950*950	32T22	1100*1100	32T22	250*900	8T18	13T18	8T18	3T18
10-story	F5	1050*1050	32T22	1300*1300	38T22	250*1000	7T22	10T22	7T22	3T22
14-story	F6	1150*1150	42T22	1400*1400	44T22	250*1150	9T22	11T22	9T22	3T22

The first yield for these frames has been accrued in displacement: 0.038m, 0.057m, 0.08m, 0.036m, 0.06m, and 0.08 m, respectively; the first yield has been accrued at base shear values of 1139 kN, 1152 kN, 1484 kN, 2232 kN, 2620 kN, and 3401 kN, respectively. The failure point of frames was defined when the compressive concrete strain of columns reached 0.004 according to the unconfined concrete model of the Mander et al. [20]. The base shear of failure was obtained (1958 kN, 2178 kN, 2925 kN, 3850 kN, 4294 kN, and 5282 kN, respectively). The base shear obtained from the equivalent static method was close to the base shear for the first yield. The slope of pushover curves is gradually reduced with an increase in the lateral displacement of the building. This is due to the progressive formation of plastic hinges in beams and ground-floor columns throughout the structure. Three PBSD methods (ATC-40, FEMA-440, and N2) were used to determine the performance points. The capacity spectrum method (ATC-40) gave the lowest performance point. However, all three methods indicate that the margin of safety against collapse according to base shear and target displacement is high and there are sufficient strength and displacement reserves. FEMA-440 and N2 methods give closed results for 10 and 14-story buildings designed for peak ground acceleration of 0.15g. In general, the displacement coefficient method (FEMA-440) gave the largest performance point, and the N2 method gives performance points located between those obtained by the capacity spectrum method and the displacement coefficient method. This result was confirmed by Hakim et al [14].





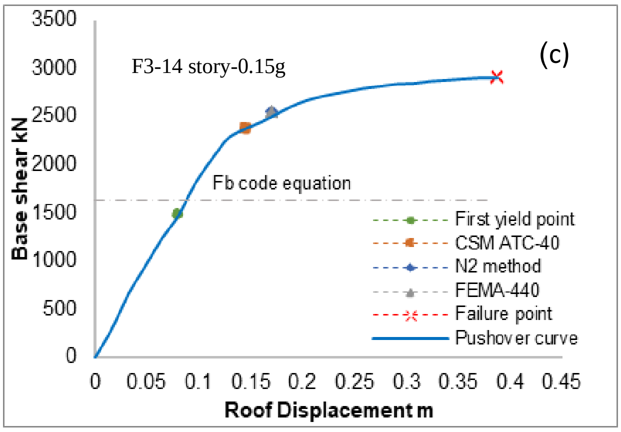


Fig.4 Pushover curve with performance point for (a) 6-story building; (b) 10-story building; (c) 14-story building; designed for peak ground acceleration of 0.15g.

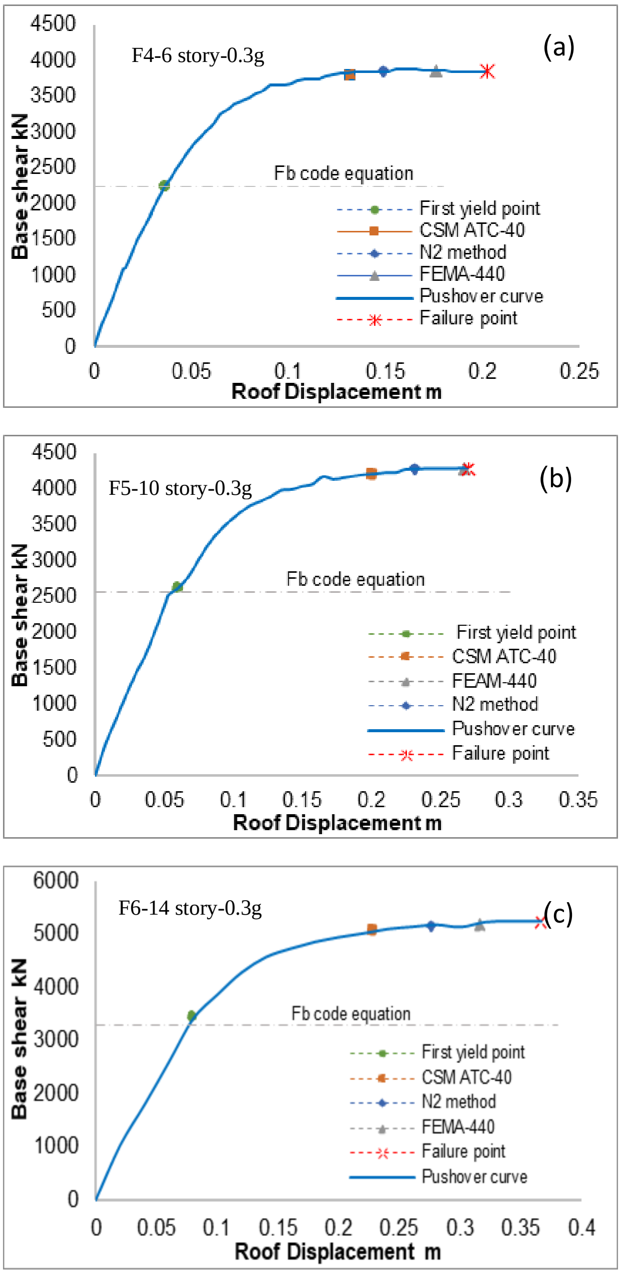
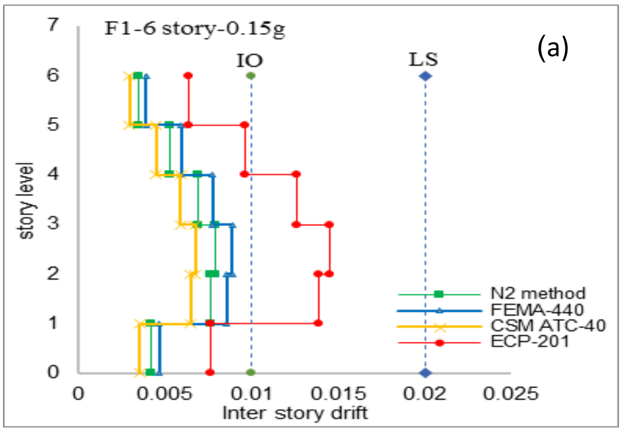
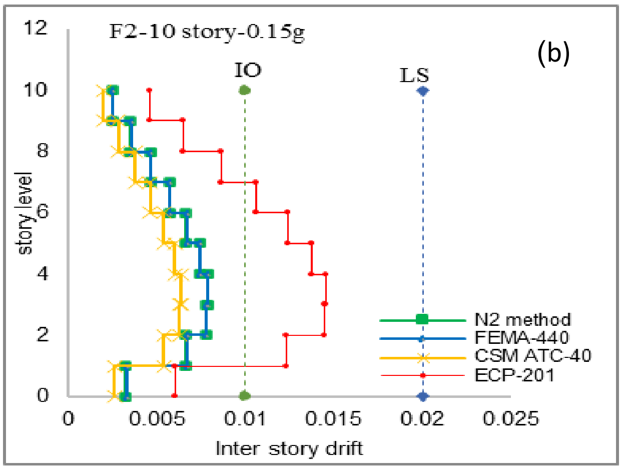


Fig.5 Pushover curves with performance points for (a) 6-story building; (b) 10-story building; (c) 14-story building; designed for peak ground acceleration of 0.3g.

Figure 6 (a) shows inter-story drift for the three buildings designed for peak ground acceleration of 0.15g at the performance point. From these results, the maximum drift ratio for a six-story building is 0.0067, 0.0088, 0.0078, and 0.0144 for the ATC-40, FEMA-440, N2, and ECP-201 methods, respectively. Fig.6 (b) shows inter-story drift at the performance point. From these results, the maximum drift ratio for a ten-story building is 0.0063, 0.0078, 0.00783, and 0.01459 for ATC-40, FEMA-440, N2 methods, and ECP-201, respectively. Fig.6 (c) shows the maximum drift ratio for fourteen stories is 0.00487, 0.00572, 0.0057, and 0.0149 for ATC-40, FEMA-440, N2 methods, and ECP-201, respectively. For all performance procedures, the maximum story drift for all buildings is < 0.01, which can be categorized as immediate occupancy (IO) performance according to Table 1. On the other hand, the performance level of frames calculated by the code equation was life safety (LS). This may be due to the inaccuracy of the drift amplification factor (0.7*R) used to elastic drift to seismic inelastic drift. It is worth mentioning that the code equivalent static methods gave conservative drift when the number of floors increased.





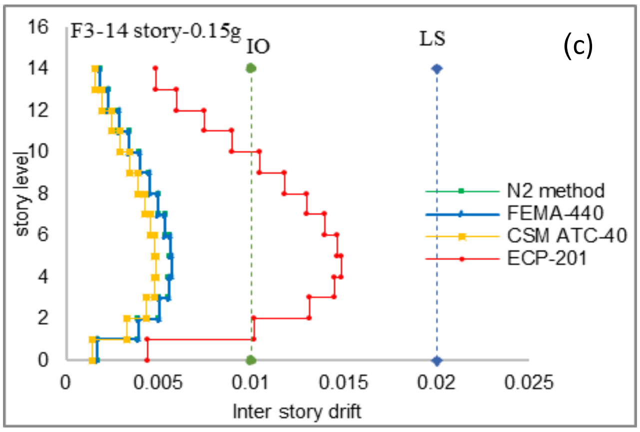
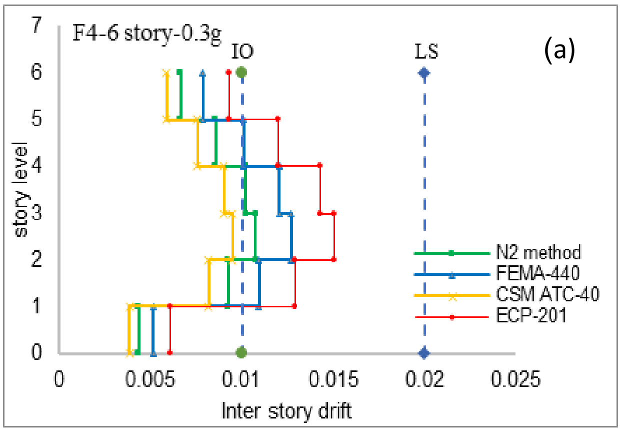


Fig. 6 The inter-story drift ratios for different methods with performance limits: for; (a) 6-story models; (b) 10-story models; (c) 14-story models; designed for peak ground acceleration of 0.15g.

Figure 7 shows the inter-story drift for the three buildings designed for peak ground acceleration of 0.3g at the performance point. From these results, the maximum drift ratio for a six-story building is 0.0094, 0.012, 0.0107, and 0.01502 for the ATC-40, FEMA-440, N2 method, and ECP-201, respectively. Fig.7 (b) shows inter-story drift at the performance point. From these results, the maximum drift ratio for ten-story buildings is 0.0091, 0.0122, 0.0106, and 0.0148 for the ATC-40, FEMA-440, N2, and ECP-201 methods, respectively. Fig.7 (c) The maximum drift ratio for a fourteen-story building is 0.0076, 0.0106, 0.0093, and 0.0148 for ATC-40, FEMA-440, N2 methods, and ECP-201, respectively. The maximum story drift for all buildings obtained from the ATC-40 is <0.01, which can be categorized as immediate occupancy (IO) according to Table 1. The maximum story drift for all buildings obtained from the displacement coefficient method is between 0.01-0.02, which can be categorized as "Life Safety" (LS) according to Table 1. The N2 method gave performance-level life safety for six and ten stories; however, it gave immediate occupancy (IO) performance for the frame of fourteen stories. On the other hand, the performance level of frames calculated by the code equation was life safety (LS). This may be due to the inaccuracy of the drift amplification factor (0.7*R) used to convert elastic drift to seismic inelastic drift. It is worth mentioning that the code equivalent static methods gave conservative drift when the number of floors increased.



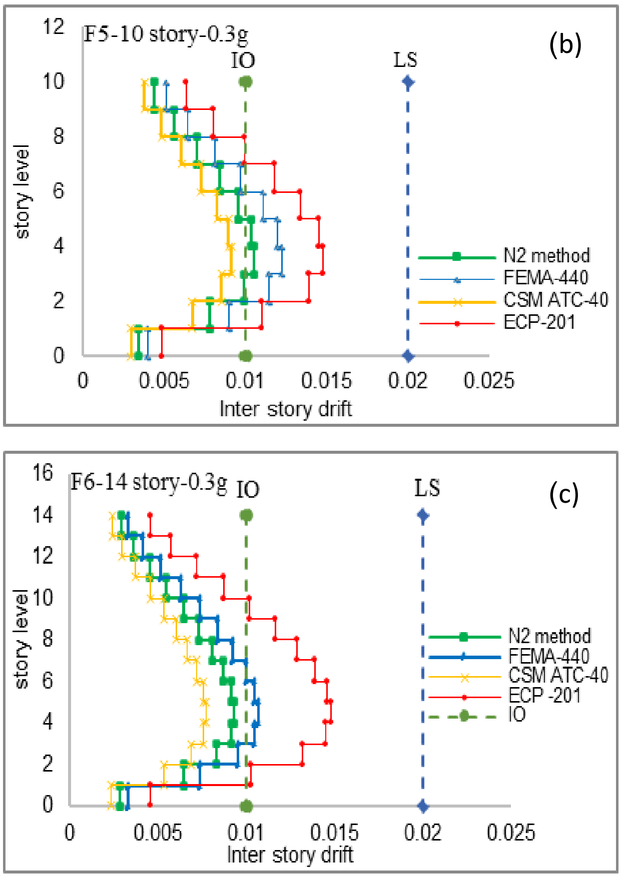


Fig.7 The inter-story drift ratios for different methods with performance limits: for; (a) 6-story models; (b) 10-story models; (c) 14-story models; designed for peak ground acceleration of 0.3g.

Figure 8 shows the accumulative story shear for the three buildings designed for a peak ground acceleration of 0.15g. Fig.8(a) shows the maximum base shear for the six-story frame is 1958 kN and 1125 kN from pushover analysis and the seismic design force, respectively. Fig.8(b) shows the maximum base shear for a ten-story frame is 2178 kN and 1278 kN from pushover analysis and the seismic design force, respectively. Fig.8(c) shows the maximum base shear for a fourteen-story frame is 2925 kN and 1636 kN from pushover analysis and the seismic design force, respectively. It is also observed that the story shear values using the triangular load pattern lead to the maximum base shear value in the building, apparently providing a conservative prediction of base shear. These results mean there is a large margin of safety between nonlinear pushover analysis and the linear equivalent static method (about 1.75–1.70) of the code equivalent static method and gives a large structural element for design, which can be optimized. In increasing the number of stories, the margin of safety according to base shear is reduced.

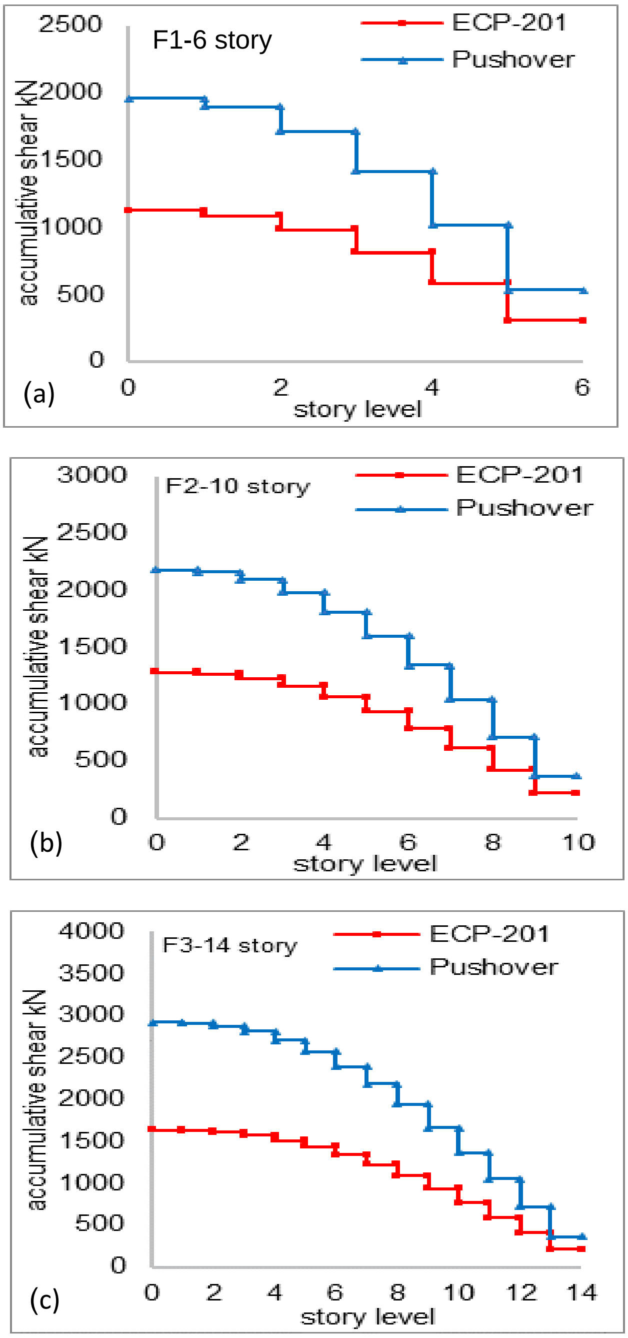


Fig.8 Accumulative story shear for (a) 6-story building; (b) 10-story building; (c) 14-story building; designed for peak ground acceleration of 0.15g.

Figure 9 shows the accumulative story shear for the three buildings designed for a peak ground acceleration of 0.3 g. Fig.9(a) shows the maximum base shear for a six-story frame is 3850 kN and 2250 kN from pushover analysis and the seismic design force, respectively. Fig.9(b) shows the maximum base shear for the ten-story frame is 4294 kN and 2557 kN from pushover analysis and the seismic design force, respectively. Fig.9(c) shows the maximum base shear for a fourteen-story frame is 5228 kN and 3272 kN from pushover analysis and the seismic design force, respectively. It is also observed that the story shear values using the triangular load pattern lead to the maximum base shear value in the building, apparently providing a conservative prediction of base shear. These results mean there is a large margin of safety between nonlinear pushover analysis and the linear equivalent static method (about 1.75–1.60) of the code equivalent static method and gives a large structural element for design, which can be optimized. In increasing the number of stories, the margin of safety according to base shear is reduced.

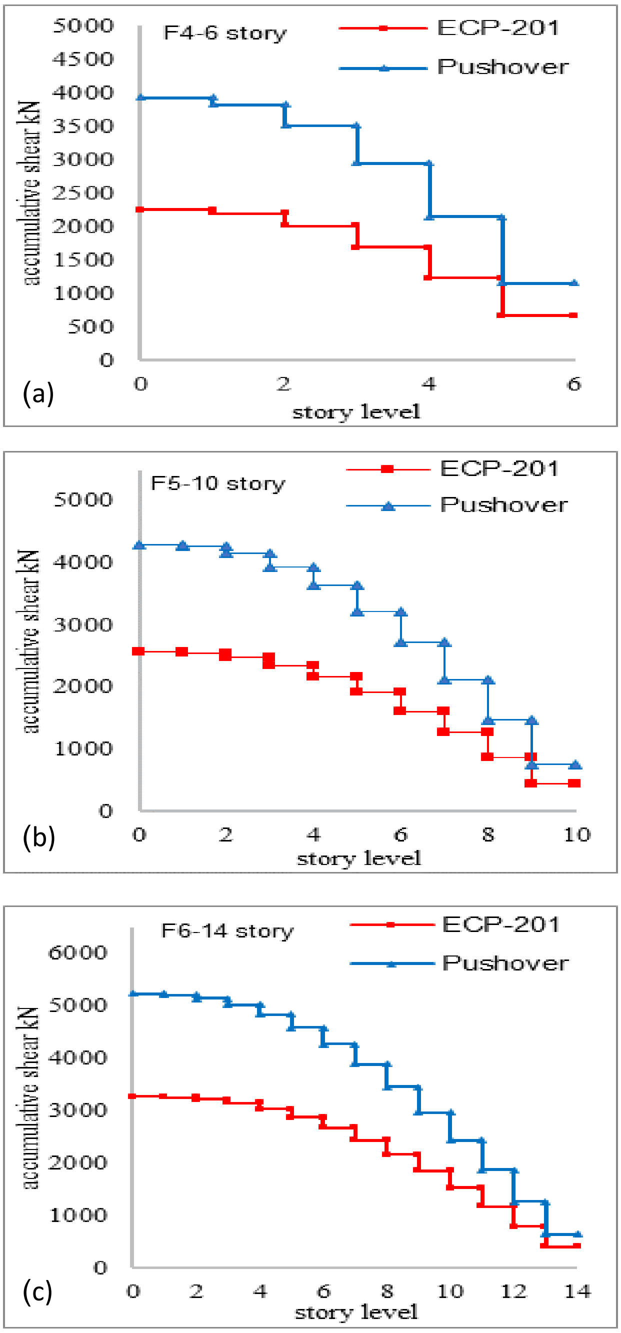


Fig. 9 Accumulative story shear for (a) 6-story building; (b) 10-story building; (c) 14-story building; designed for peak ground acceleration of 0.3g.

4. Conclusions

This study aimed to define the seismic performance of RC regular frames using nonlinear performance-based design methods. In this study, two groups of RC buildings in two different seismic zones have been studied: the first group contained six, ten, and fourteen-story frames designed for peak ground acceleration of 0.15g, and the second group contained six, ten, and fourteen-story frames designed for peak ground acceleration of 0.3 g. These frames were designed based on the ECP-201 code [18]. The performance methods specified in ATC-40[2], N2[3], and FEMA-440[4] methods were used to define the performance point and performance level of the frame.

The main conclusions of this study are as follows:

1-According to performance structural levels concerned with the lateral drifts, the maximum inter-story drift ratio for all buildings designed according to ECP-201code[18] for peak ground acceleration in seismic zones 0.15g and 0.3g is expected to be between (0.01-0.02), which can be classified in life safety performance level according to FEMA-440 [4] specifications.

2-According to the performance point of view, the base shear at the performance point is more than the design base shear obtained from the code equivalent static method. This confirmed that the code design is safe and conservative, but it can be optimized. By increasing the peak ground acceleration, the margin of safety according to base shear is slightly reduced.

3-According to inter-story drift values obtained by the ECP -201 [18] seismic code equation, the performance level was life safety (LS). On the other hand, the performance level obtained by (PBSD) in the case of peak ground acceleration of 0.15g was immediate occupancy (IO). This may be due to the inaccuracy of the drift code amplification factor (0.7*R) used to convert elastic drift to seismic inelastic drift. It is worth mentioning that the code equivalent static methods gave conservative drift because it is based on simple linear assumptions.

4-The capacity spectrum method (ATC-40) [2] usually underestimated the drift demand, and the displacement coefficient method (FEMA-440) [4] overestimated the drift in most cases.

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