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Seismic performance of existing R.C. framed buildings

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KEYWORDS

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Abstract The earthquakes disasters basically occur due to buildings damage not because of the earth shaking. Therefore, the countries have being updated the seismic codes. The seismic loads for buildings design in Egyptian Code have been changed from (EC-1994) to (ECP-201, 2012). On the other hand, the need is raised to study the vulnerability of existing buildings, which can be divided into the buildings designed to resist the gravity loads only (GLD) and the buildings designed according to Egyptian code (EC-1994). Comparison between forces due to Egyptian code for loads (EC-1994) and (ECP-201, 2012) is carried out on the multi-stories R.C. framed buildings which are the most common type of existing buildings in Egypt. To investigate the vulnerability of existing buildings, nonlinear static pushover analysis is conducted to evaluate the real strength of the existing buildings. Moreover, it is considered a useful and effective tool for the performance of three framed buildings: 3, 6 and 10 stories due to expected future earthquakes. Finally, it is found that the vulnerability of existing GLD buildings occurs at expected ground accelerations (a_g) greater than 0.125 g in Egyptian seismic map, while the EC-94 designed buildings behave elastically up to (a_g) equals to 0.2 g and above that a slight damage may occur.

Introduction

Structural design should achieve high probability of survival under the expected loads. The earthquake loadings are different from other loads because of the high deformations and stresses conducted under earthquake effect. Codes require that structures possess adequate ductility to allow them to dissipate most of energy from the ground motions through inelastic deformations. This concept prevents the buildings from collapse even if it is seriously damaged because it is generally uneconomical to design most buildings to respond elastically to moderate-to-strong earthquakes [1]. Therefore, it has become an urgent issue to achieve seismic hazard mitigation of existing buildings that were designed according to gravity loads only or designed according to earlier codes. Potential structural deficiencies in the existing structures are assessed by the code seismic-resistant design and pushover approaches. In the first approach, the potential deficiencies are determined

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by redesigning under one selected seismic combination code in order to show which members would require additional reinforcement. The second approach is a nonlinear pushover analysis [2]. The nonlinear pushover analysis is a simple technique to predict the seismic response of buildings and evaluate the adequacy of the lateral strength of the buildings. A pushover analysis is performed by subjecting a structure to monotonically increasing pattern of lateral forces, representing the inertia forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads, various structural elements yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness [3]. It is useful for the methodology of performance-based seismic engineering introduced by ATC-40 [4]. The modern seismic codes and guideline documents support the application of pushover procedure as a practical and effective approach to attain performance-based seismic assessment of multistory frame buildings [5]. Seismic performance is described by designating the maximum allowable damage state for an identified earthquake ground motion. Performance objectives such as life safety, collapse prevention, or immediate occupancy are used to define the state of the building following a design earthquake [6]. The performance-based roots of ATC-40 are essentially the same as FEMA-273 [7] and FEMA-274 [8], and SEAOC [9].

The objective of this research is to predict earthquake risk of the existing framed buildings. In this study, the potential structural deficiencies in the existing frames are determined due to lateral loads according to Egyptian code (ECP-201, 2012) to detect potential weak locations in the structures. Pushover analysis is conducted to know the sequence of cracking, yielding and failure on the members and structure. Moreover, ATC-40 approach is carried out to study the seismic performance of GLD and EC-94 designed moment resisting frames.

Analysis procedure

Recent trends in earthquake engineering practice aim to providing better seismic assessment for building structures through explicit consideration of the inelastic performance of the building [5]. The practical objective of inelastic seismic analysis procedures is to predict the behavior of the structure in future earthquakes, which is important in existing building. The pushover analysis is conducted on two dimensional R.C. frames by using a Computer Program for Inelastic Damage Analysis of R.C. Structures (IDARC version 6) [10]. The computer program IDARC was conceived as a platform for nonlinear structural analysis in which various aspects of concrete behavior can be modeled, tested and improved upon. Program developed and enhancements have been primarily to link experimental research and analytical developments. One of the significant features incorporated in the program, to implement inelastic behavior in macro-models, is the distributed flexibility model that replaced the commonly used hinge model developed for steel frames. The hinge model is not suitable for R.C. elements since the inelastic deformation is distributed along the member rather than being concentrated at critical sections. To trace the hysteretic response of a section, a three parameter model was developed. Through the combination of three basic parameters and a trilinear skeleton curve stiffness degradation, strength deterioration and pinching response can be modeled. Hysteretic behavior is specified at both ends of each member and it changes from one linear stage to another, depending on the history of deformations.

The building is modeled as a series of plane frames linked by a rigid horizontal diaphragm. Each frame is in the same vertical plane, and no torsional effects are considered. The program calculates the forces, deformations and the damage index which represent the frame state under earthquake excitation. The program uses a distributed flexibility model in constructing the element stiffness matrix leading to including the effect of spread plasticity. Column elements were modeled considering macro-models with inelastic flexural deformations, and elastic shear and axial deformations. Beam elements are modeled using a nonlinear flexural stiffness model with linear elastic shear deformations considered. Column and beam elements include a rigid length zone to simulate the increase in the stiffness of the element. Parameters used in the nonlinear analysis are the stiffness properties and inelastic hysteretic properties which are based on the default values defined in the program.

The predicted structures may not be as the same as the actual observed performance of such structures after earthquakes because the models used in this study neglect foundation flexibility and many other elements that contribute to their strength such as infilled walls. The analysis of multistories reinforced concrete frame structures under static loads has been carried out using Sap2000 software, finite element package. According to Egyptian code (ECP-201, 2012), the flexural rigidity of R.C. columns is assumed as 0.7 E_cI_g and the flexural rigidity of R.C. beams is assumed as 0.5 E_cI_g, where E_c is Young's modulus for concrete and I_g is the moment of inertia of gross concrete section.

Description of considered cases

The moment resisting frame system consists of beams and columns in which bending of these members provides the resistance of lateral forces. Frame structures are modeled as 3, 6 and 10 stories plane frames with 3 bays to represent typical low-to-moderate rise moment-resisting reinforced concrete framed residential buildings located in Egypt. The typical height of all floors is 3 m except for the ground story, which is 4 m. The total heights of the three buildings are 10, 19 and 31 m, respectively.

Three interior frames are fully designed for gravity loads to represent the GLD buildings. The cross sections and reinforcement of beams of the studied frames are given in the Table 1. Also, Table 1 shows the cross sections of columns, where the reinforcement ratio of columns is 1% from the total cross section. The columns and beams have the same cross sections throughout the height of the frames. The columns are considered fixed at the base for the considered frames. The compressive strength of concrete is 250 kg/cm^2 while the yield strength of reinforcement steel is 3600 kg/cm^2. The direction of ground motion is seldom correlated with the orientation of the building. The worst case is usually considered when the ground shakes parallel with each Cartesian axis of the building. The system is modeled as two separate planer systems, each of which is subjected to all earthquake forces.

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Table 1 Dimensions and reinforcement of gravity load-designed R.C. frames.

3, 6, 10 GLD stories frames	Sec. (mm)	Exterior beam reinforcement				Interior beam reinforcement			
		Exterior edge (%) & As		Interior edge (%) & As		Left edge (%) & As		Right edge (%) & As	
	250×500	Top	Bot.	Top	Bot.	Top	Bot.	Top	Bot.
		0.18 $2\varphi12$	0.18 $2\varphi12$	0.36 $4\varphi12$	0.18 $2\varphi12$	0.36 $4\varphi12$	0.18 $2\varphi12$	0.36 $4\varphi12$	0.18 $2\varphi12$

		Exterior column	Interior column
3 stories frame	Sec. (mm)	250×300	250×400
6 stories frame	Sec. (mm)	250×500	250×700
10 stories frame	Sec. (mm)	250×700	250×1200

The direction of columns depth should be distributed into two directions to resist earthquakes, but sometimes this concept cannot be achieved in gravity load designed buildings. The effect of varying the stiffness of columns in earthquake direction is studied. Three cases are considered. In case 1, all columns are used in strong direction (columns depth is in the direction of earthquake forces). In case 3, all columns are used in weak direction (columns width is in the direction of earthquake forces) while in case 2, columns depth is distributed in two directions (zigzag shape).

A comparison between seismic egyptian code (EC-1994) And (ECP-201, 2012)

Egyptian code (EC-1994) [11] uses equivalent horizontal static forces, based on empirical formulas to predict inertia forces due to earthquakes. Egyptian code (ECP-201, 2012) [12] uses response spectrum to represent the earthquake shaking and it allows for using the equivalent static loads to represent the seismic loads for regular structures. The lateral forces from (ECP-201, 2012) are ultimate forces while lateral forces from (EC-1994) are working forces. Therefore, (ECP-201, 2012) lateral loads were divided by 1.4 for comparison purpose. The lateral base shear for (EC-1994) is calculated as follows:

$$V = ZISKCW \tag{1}$$

Z is seismic zone coefficient which represents the maximum effective peak ground acceleration, the Egyptian seismic map consists of three regions, I is an important factor, S is a numerical coefficient for site-structure resonance, K is a factor depending on the structural system. C is a factor depending on the natural frequency of the structure ($=1/15\sqrt{T}$), where T is the natural period of the structure in seconds, T = 0.1 N for moment resisting frame system where N is the number of stories. W is the total permanent load.

The lateral base shear for (ECP-201, 2012) is calculated according to the following equation

$$F_b = S_d(T)\cdot\lambda W/g \tag{2}$$

$$T_C \leqslant T \leqslant T_D: \quad S_d(T) = a_g\gamma_1 S\frac{2.5}{R}\left[\frac{T_C}{T}\right]\eta \tag{3}$$

a_g is the peak ground acceleration which varies from 0.05 g to 0.3 g in Egyptian contours map. T is the natural period of the structure in seconds which is equal to $C_t(H)^{3/4}$. S is the soil coefficient, R is the force reduction factor, W is the total permanent load in addition to 25% of live load. The comparison of base shear obtained by the code formula (ECP-201, 2012) and (EC-1994) for 3, 6 and 10 stories R.C. concrete frames is shown in Table 2. The assumed parameters of comparison are listed below.

Table 2 Base shear obtained by (ECP-201, 2012) and (EC-1994).

	EC-1994		ECP-201, 2012		ECP201, 2012/EC-1994
	Z	V/W	a_g (g)	V/W	
3 stories frame	0.1	0.011	0.05	0.013	1.18
	0.2	0.022	0.1	0.028	1.27
	0.3	0.033	0.15	0.043	1.3
	0.3	0.033	0.2	0.056	1.7
	0.3	0.033	0.25	0.07	2.12
6 stories frame	0.1	0.008	0.05	0.0104	1.3
	0.2	0.0156	0.1	0.021	1.35
	0.3	0.023	0.15	0.0311	1.35
	0.3	0.023	0.2	0.041	1.78
	0.3	0.023	0.25	0.051	2.21
10 stories frame	0.1	0.006	0.05	0.0075	1.25
	0.2	0.012	0.1	0.015	1.25
	0.3	0.018	0.15	0.022	1.22
	0.3	0.018	0.2	0.03	1.67
	0.3	0.018	0.25	0.037	2.06

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For EC-1994: Z is equal to 0.1, 0.2 and 0.3 for regions 1, 2 and 3 respectively. S is equal to 1.15 for moderate dense soil. I is equal to 1 for residential buildings. K is equal to 0.8 for non-ductile R.C. frames.

For (ECP-201, 2012): Five levels of seismic ground motions were considered, (a_g = 0.05 g, 0.1 g, 0.15 g, 0.2 g and 0.25 g). Type (1) of response spectrum was used with moderate dense soil

$$S = 1.5 \quad T_B = 0.1 \quad T_C = 0.25 \quad TD = 1.2 \quad \gamma_1 = 1 \quad \eta = 1$$

C_t is equal to 0.075. R is equal to 5 for moment resisting R.C. frame system with limited ductility.

The following notes may be conducted from Table 2:

(1) The values of base shear forces from ECP-201 formula are greater than EC-1994 values by about 25% in average for (a_g = 0.05–0.15 g).
(2) The values of base shear forces from EC-201 formula are greater than EC-1994 values by about 67–121% for a_g = 0.2–0.25 g.
(3) With increasing the number of stories, the lateral loads to total weight decease for both Codes.
(4) At maximum peak seismic zone, the lateral loads V/W (EC-1994) are 0.033, 0.023 and 0.018 for 3, 6 and 10 stories frames, respectively, where V/W (ECP-201, 2012) are 0.07, 0.051 and 0.037 for 3, 6 and 10 stories frames, respectively.

Table 3 illustrates the effect of both codes on the design of the model framed buildings. EC-94 refers to frames that were designed according to the Egyptian code of practice for design and construction of reinforced concrete structures (ECCS-203, 1995) where the seismic loads were calculated according to EC-1994 (Z = 0.3). On the other hand, EC-2012 refers to frames that were designed according to the Egyptian code for design and construction of reinforced concrete structures (ECP-203, 2007). The seismic loads were calculated according to (ECP-201, 2012) at zone 5a, the design ground acceleration of this zone is 0.25 g.

The reinforcement details of R.C. elements of frames with limited ductility according to both codes require to increasing the stirrups of beams and columns at their ends, where the column stirrups extend through beam column connection. Furthermore, the top and bottom longitudinal reinforcements of beams extend through the connections with sufficient development length.

Table 3 illustrates the cross section of beams and columns and the reinforcement of beams. The depth and longitudinal reinforcement of beams of EC-94 frames increase in relation to GLD frames. The significant change is found in the beams of EC-2012 frames. The columns of EC-94 frames are similar to columns of GLD frames with width 30 cm instead of 25 cm. The depth of columns of EC-2012 frames increases by about 10%, where the reinforcement percentage of columns of all frames is 1% from the total cross section.

Effect of lateral forces due to (EC-201, 2012) on the existing framed buildings

Every structural system is designed to have a seismic capacity that exceeds the anticipated seismic demand. In this study, the potential structural deficiencies in the GLD and EC-94 designed frames are determined by an analysis of these frames under seismic loads due to (ECP-201, 2012) to show which members would be over stressed. Interior beam and column at ground floor are considered as indication of all frame elements.

The bending moment is selected to represent the straining actions of frame members. The ratio M_d/M_c is used to show the ability of frame members to resist the code results. M_d is the moment demand of the members due to (ECP-201, 2012) loads, M_c is the maximum moment that members can sustain. The moment demand (M_d) related to moment capacity (M_c) of frames components is shown in Table 4.

It is evident from Table 4 that the beams at first story of GLD frames sustain to moment demand due to (ECP-201, 2012) forces more than the capacity of beams. The lateral forces in area of high ground acceleration cause increasing of M_d/M_c of beams which attain to more than 3 times, while the increasing of ratio M_d/M_c of columns amounts to 75%. Also, Table 4 illustrates that increasing the number of stories with low moment of inertia in earthquake direction may cause inelastic actions of frames due to high moment demand related to the moment capacity. Therefore, insufficient strength of

Table 3 Dimensions and reinforcement of beams of EC-94 and EC-2012 designed R.C. frames.

		Sec. (mm)	Exterior beam Reinforcement				Interior beam reinforcement			
			Exterior edge (%) & As		Interior edge (%) & As		Left edge (%) & As		Right edge (%) & As	
			Top.	Bot.	Top.	Bot.	Top.	Bot.	Top.	Bot.
3 stories	EC-94	250 × 500	0.32 2φ16	0.32 2φ16	0.64 4φ16	0.32 2φ16	0.64 4φ16	0.32 2φ16	0.64 4φ16	0.32 2φ16
	EC-2012	250 × 500	0.64 4φ16	0.32 2φ16	0.8 5φ16	0.64 4φ16	0.8 5φ16	0.64 4φ16	0.8 5φ16	0.64 4φ16
6 stories	EC-94	250 × 500	0.40 2φ16 + 1φ12	0.32 2φ16	0.64 4φ16	0.32 2φ16	0.64 4φ16	0.32 2φ16	0.64 4φ16	0.32 2φ16
	EC-2012	250 × 700	0.46 4φ16	0.46 4φ16	0.69 6φ16	0.46 4φ16	0.69 6φ16	0.46 4φ16	0.69 6φ16	0.46 4φ16
10 stories	EC-94	250 × 700	0.34 3φ16	0.34 3φ16	0.46 4φ16	0.34 3φ16	0.46 4φ16	0.34 3φ16	0.46 4φ16	0.34 3φ16
	EC-2012	250 × 700	0.46 4φ16	0.46 4φ16	0.69 6φ16	0.46 4φ16	0.69 6φ16	0.46 4φ16	0.69 6φ16	0.46 4φ16

Table 4 Effect of forces due to (ECP-201, 2012) on GLD frames.

	3 stories			6 stories			10 stories		
	Case 1	Case 2	Case 3	Case 1	Case 2	Case 3	Case 1	Case 2	Case 3
a_g (g)	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c
Interior beam									
0.05	0.89	0.92	0.9	0.97	1.11	1.1	1.09	1.18	1.32
0.1	1.16	1.2	1.16	1.34	1.58	1.5	1.47	1.57	1.84
0.15	1.42	1.53	1.42	1.65	2	1.92	1.74	2	2.36
0.2	1.66	1.84	1.65	2.1	2.5	2.32	1.97	2.3	2.9
0.25	1.86	2	1.86	2.4	2.87	2.68	2.37	2.9	3.39
Interior column									
0.05	0.28	0.33	0.34	0.2	0.32	0.36	0.14	0.25	0.27
0.1	0.53	0.67	0.7	0.38	0.61	0.72	0.28	0.48	0.57
0.15	0.8	1.03	1.03	0.6	0.93	1.1	0.43	0.71	0.85
0.2	1.07	1.37	1.4	0.78	1.22	1.45	0.6	0.88	1.13
0.25	1.31	1.6	1.68	0.93	1.5	1.75	0.7	1.16	1.44

Table 5 Effect of forces due to (ECP-201, 2012) on the EC-94 frames.

	3 stories	6 stories	10 stories	3 stories	6 stories	10 stories
	Interior Beam			Interior column		
a_g (g)	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c	M_d/M_c
0.15	0.77	1.19	0.88	0.69	0.57	0.42
0.2	0.91	1.47	1.01	0.92	0.76	0.52
0.25	1.0	1.7	1.26	1.08	0.92	0.69

R.C. elements of the GLD buildings, in addition to the absence of special provisions to insure the ductility, both can cause unexpected structural failure during stronger earthquakes. It is indicated that the R.C. elements in low-to-moderate rise GLD buildings are not adequate to achieve safety according to (ECP-201, 2012) lateral loads especially in high seismic regions. It is important to illustrate that, Code lateral forces used for seismic evaluation of existing buildings are reduced by multiplying a factor of 0.75. This reduced force level is justifiable because an existing building does not need to have the same level of safety factor as a new building since the remaining useful life of an existing building may be less than that of a new building [13].

Table 5 shows the ratio of M_d/M_c which used to illustrate the effect of lateral loads due to (ECP-201, 2012) on the EC-94 frames. M_d is the moment demand of the members due to (ECP-201, 2012) loads, M_c is the maximum moment that members can sustain due to load combination ($0.9D + S$) where D = dead loads and S = seismic loads due to EC-1994 ($Z = 0.3$).

It is concluded from Table 5 that the M_d/M_c decreases with increasing the number of stories due to the large cross section of columns. At 0.2–0.25 g, the seismic loads due to (ECP-201, 2012) cause additional stresses above than capacity of beams members.

Structural capacity

Capacity is a function of strength, stiffness and deformability conjectured by the system configuration and material properties of the structure. The nonlinear static pushover analysis is conducted to create the capacity curve of structures when

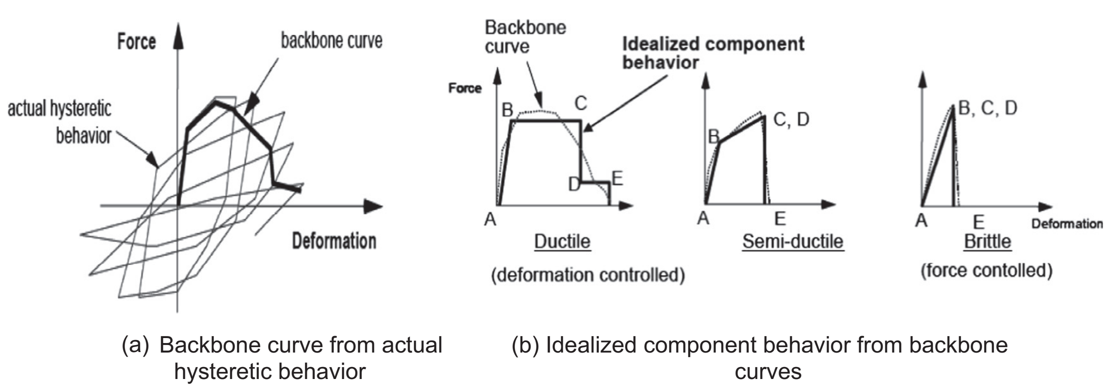

Fig. 1 Idealized component force–deformation relationships [14,15].

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subjected to lateral forces. It is generated by subjecting a detailed structural model to one or more lateral load patterns (vectors) and then increasing the magnitude of the total load in a step-by-step and the corresponding incremental displacement is calculated to generate a nonlinear inelastic force–deformation relationship for the structure at a global level. The load vector is usually an approximate representation of the relative accelerations associated with the first mode of vibration for the structure. The results from pushover analyses are presented in graphs that describe the variation of base shear versus top displacement. Capacity curve defines the capacity for an assumed force distribution and displacement pattern, if the building displaces laterally, its response must lie on this capacity curve. A point can be found on the capacity curve that estimates the maximum displacement of the building when the earthquake will cause and defines a specific damage state for the structure. Pushover technique allows the sequence of cracking, yielding and failure on the members and structure and it is benefit to highlight potential weak regions in the structure. Detailed structural models for inelastic analysis are normally based on approximations derived from the test results on individual components, Fig. 1 [14,15].

The pushover analysis may be carried out using force control or displacement control. In the former option, the structure is subjected to an incremental distribution of lateral forces and the corresponding incremental displacement is calculated. In the latter option, the structure is subjected to a displacement profile, and the lateral force required to generating that deformation is calculated. Force controlled actions provide little deformation to the entire building through inelastic behavior, inelastic action in these elements may cause a sudden or total collapse of the structures [10]. In this study, the force control option is used to model frames, because of the limited ductility associated with force control.

Capacity curves of GLD frames

To investigate the lateral capacity of GLD buildings, the nonlinear pushover analysis is carried out on 3, 6 and 10 stories GLD frames (case 2). Capacity curves of 3, 6 and 10 stories GLD frames are shown in Fig. 2. The maximum values of lateral load related to the total weight (V/W) are of low values for GLD frames which is obvious in 6, 10 stories, the maximum values of (V/W) are equal to 0.064, 0.02 and 0.01 for the 3, 6 and 10 stories frames, respectively. The pushover analysis illustrates that the beams start from cracking to yielding at low level of lateral loads. This is attributed to the GLD frames dominated by weak beams due to low reinforcement at end beams. Therefore, the GLD buildings tend to be of brittle behavior and are more vulnerable with increasing the number of stories.

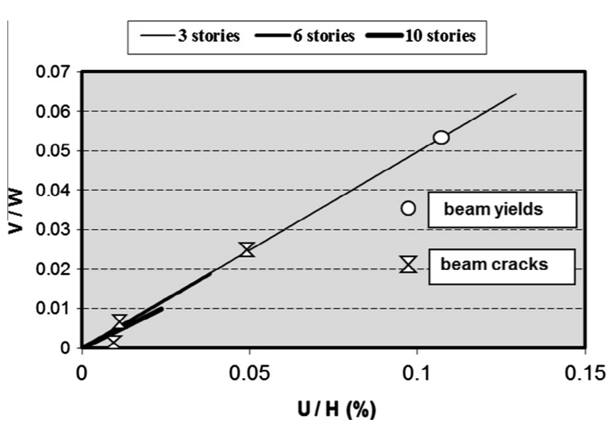

Fig. 2 Capacity curves of gravity load designed GLD frames.

Capacity curves of EC-94 designed frames

Fig. 3 illustrates the capacity curves of 3, 6 and 10 stories frames which have been designed to resist seismic loads from

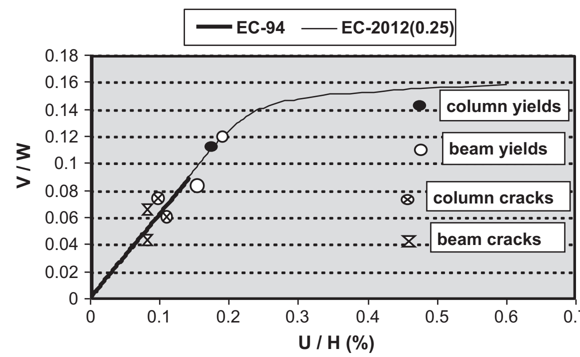

Fig. 3a Capacity curves of 3 stories frames designed according to Egyptian Codes.

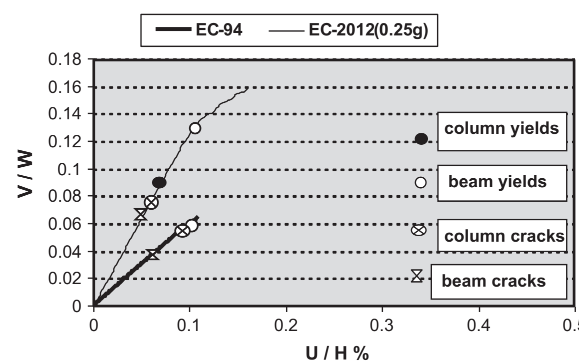

Fig. 3b Capacity curves of 6 stories frames designed according to Egyptian Codes.

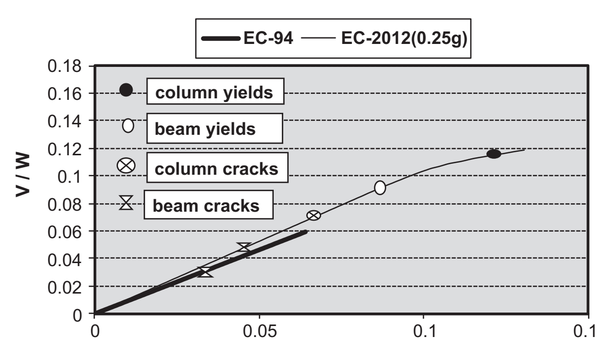

Fig. 3c Capacity curves of 10 stories frames designed according to Egyptian Codes.

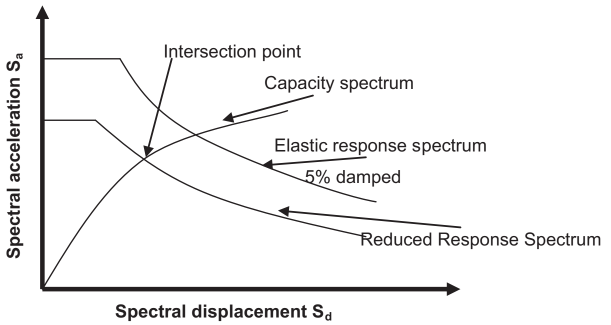

Fig. 4 Capacity spectrum procedure to determine performance point.

(EC-1994) ($Z = 0.3$) and compared to frames designed according to seismic loads from (ECP-201, 2012) (0.25 g). The curves show the good behavior of frames designed according to codes compared to GLD frames. The plotted relationship can be divided into 2 stages, the response of frames is mainly elastic up to yielding of frame elements and after that the relationship is curved. It is noticed from Fig. 3 that, the capacity curves of EC-94 frames reach to high values of V/W more than GLD frames by about 1.5, 3.25 and 6 times for 3, 6 and 10 stories frames, respectively. Also, the maximum values of V/W of EC-2012 frames are greater than values of EC-94 frames. It is found from Fig. 3 that, cracks are anticipated in R.C. elements of EC-94 frames when base shear to total weight (V/W) equals to 0.042, 0.024, and 0.032 for 3, 6 and 10 stories frames, while cracks start in R.C. elements at 0.064, 0.066, and 0.047 for 3, 6 and 10 stories EC-2012 frames, respectively.

Performance-based seismic engineering

The performance-based seismic engineering (PBS) is a methodology introduced by ATC-40 [4] for seismic evaluation of existing buildings and it is an attempt to predict the maximum allowable damage state of building for an identified seismic hazard. One of the methods used to determine the performance point is the capacity spectrum method. The demand and capacity parameter for the analysis is the lateral displacement of the building. The capacity spectrum method requires that both capacity curve and the demand curve be represented in response spectral ordinates. The seismic demand on a structure is usually expressed in the form of a design spectrum according to the prevailing seismic code and including all structural and zoning parameters. The capacity curve produced is plotted against the demand response spectrums. The intersection of the demand spectrum with the nonlinear pushover response is called "Performance Point". It corresponds to the expected state the structure under the considered earthquake. Depending on the position and state of the performance point (with respect to the actual pushover curve), the analyst may

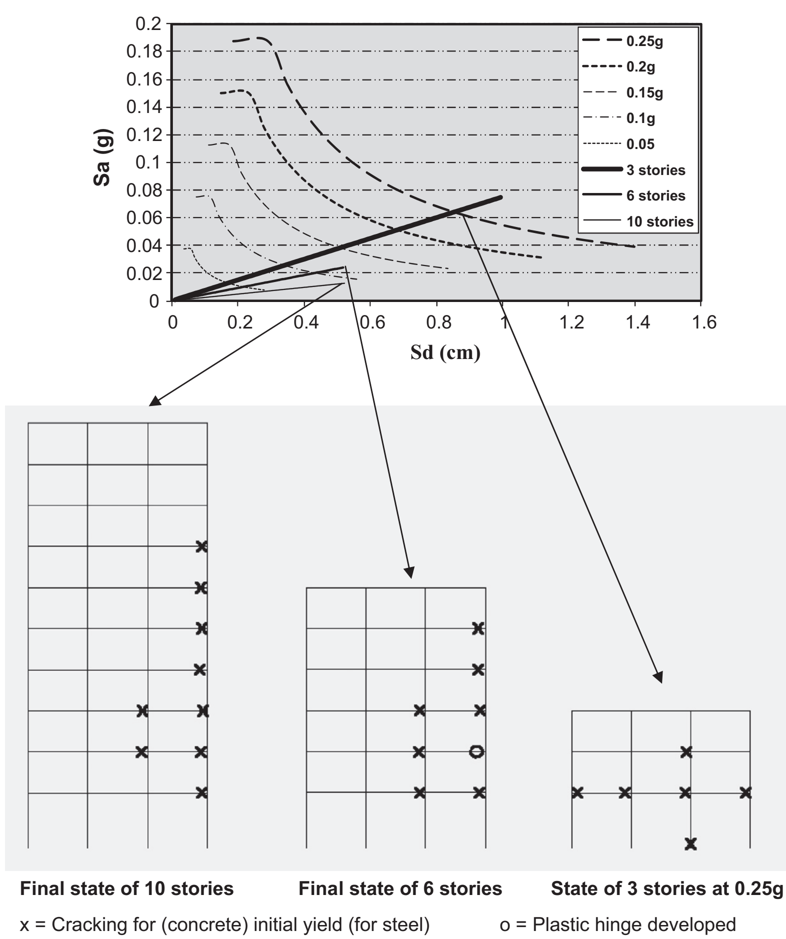

Fig. 5 Seismic performance points and state of the GLD frames.

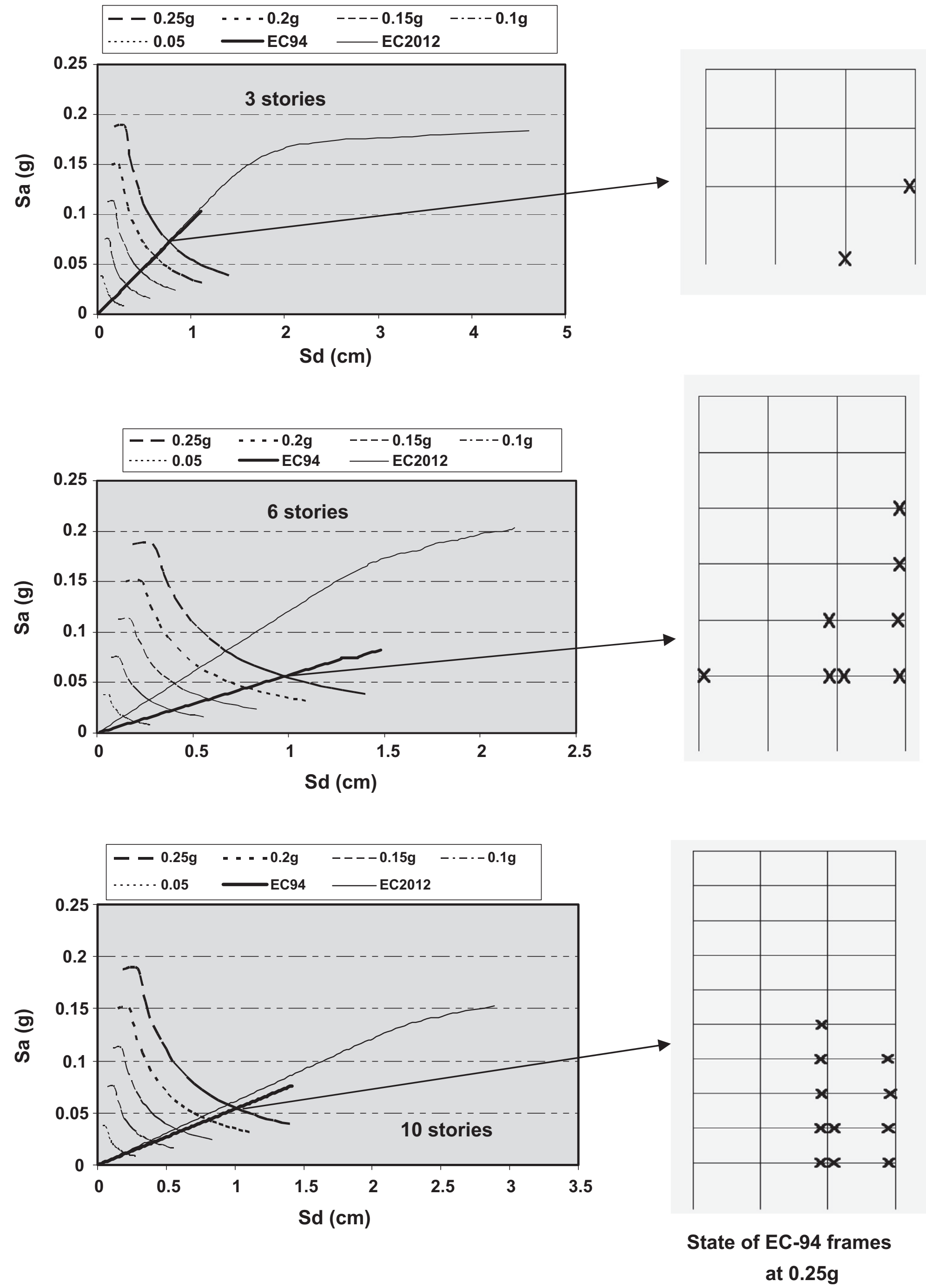

Fig. 6 Seismic performance points of the EC-94 (Z = 0.3) state of EC-94 frames & EC-2012 (0.25 g) at 0.25 g.

decide on how safe or vulnerable the structure is and where possible strengthening should be performed [4,6]. If the demand curve intersects the capacity curve near the elastic range, then the structure has a good resistance. If the demand curve intersects the capacity curve with little strength and deformation capacity, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse [16]. The technique requires that both the demand response spectra and structural capacity curves be plotted in the spectral acceleration vs. spectral displacement domain.

To convert a spectrum from the standard S_a vs. T format found in the building code to S_a vs. S_d format. The value of S_{di} for each point on the curve of S_{ai}, T_i can be done with the equation:

$$S_{di} = T_i^2 S_{ai} g / 4\pi^2 \tag{4}$$

The capacity spectrum is developed from the capacity curve. Any point V_i, $U_{(\mathrm{roof})}$ on the capacity curve is converted to the corresponding point S_{ai}, S_{di} on the capacity spectrum using the equations:

$$S_{ai} = (V_i/W)/\alpha_1 \quad (5)$$

$$S_{di} = U/(\mathrm{PF}_1 \times \phi_{1,\mathrm{roof}}) \quad (6)$$

where α_1 and PF_1 are the modal mass coefficient and participation factors for the first natural mode of the structure, $\phi_{1,\mathrm{roof}}$ is the roof level amplitude of the first mode. For regular buildings with uniform mass and straight line mode shape, α_1 and PF_1 are equal to 0.86, 1.3 for 3 stories frame and equal to 0.78, 1.4 for 6 and 10 stories frames [4] (see Fig. 4).

Seismic performance of existing buildings GLD and EC-94 frames

To identify the seismic vulnerability of existing framed buildings under the seismic ground motions, PBS procedure is carried out on the GLD and EC-94 frames. The seismic demand is expressed in the form of a design spectrum according to (ECP-201, 2012) for 0.05 g, 0.1 g, 0.15 g, 0.2 g and 0.25 g. The performance of GLD frames (case 2) is achieved by the intersection of the demand and capacity spectrum curves, as shown in Fig. 5. Also, Fig. 5 illustrates the state of GLD frames by using nonlinear dynamic computer program IDARC.

It is found from Fig. 5 that the lateral strength of 3 stories GLD frame is adequate to resist seismic forces due to ground acceleration in all regions in Egyptian seismic map (a_g = 0.05–0.25 g). With increasing the number of stories, the lateral strength capacity can resist peak horizontal acceleration equals to 0.12 g 0.09 g for 6 and 10 frames. Above these values the probability of failure may exist due to high moment demand related to the moment capacity.

Fig. 5 shows that 3 stories GLD frame is exposed to cracks in columns and beams at ground acceleration equals 0.25 g. The final states of 6 and 10 stories GLD frames illustrate that the beams are exposed to crack and yielding due to low reinforcement at beam ends. This indicates that the GLD frames will behave poorly under high horizontal ground acceleration. Therefore, these buildings need to be retrofitted to avoid major future damage or collapse.

Fig. 6 shows seismic performance of EC-94 frames compared to EC-2012 frames. The performance points of EC-94 frames are illustrated in Fig. 6 by intersecting the demand curves with the capacity curves of frames. By correlating to actual pushover curves, it is noticed that the demand curves tend to intersect the capacity curves in the elastic range up to 0.2 g. Above that value cracks in frames elements will be noticed. Fig. 6 shows the state of EC-94 frames at 0.25 g. Beam cracks start at beam ends due to the low values of loads from Egyptian code (EC-1994) in high seismic regions. The columns still behave elastically because the cross section of columns is enough to resist lateral loads. On the other hands, performance points of the EC-2012 frames indicate that the frames behave elastically at all regions in Egyptian seismic map.

Conclusions

For the studied frames

(1) The values of base shear of low-to-moderate rise framed building from EC-2012 formula are greater than EC-1994 values by about 25% in average for a_g from 0.05 to 0.15 g. The difference increases from 67% to 121% for 0.2 g to 0.25 g.
(2) Out of nonlinear pushover analysis, it is found that GLD frames are dominated by weak beams due to insufficient longitudinal reinforcement at top and bottom beam ends so beams become more susceptible to damage, especially in 6, 10 stories frames. On the other hands, frames designed according to Egyptian code have a high capacity to resist earthquakes.
(3) Application of performance-based seismic approach ATC-40 on GLD framed buildings illustrates that the lateral strength of 3 stories frame is adequate to resist seismic forces due to ground acceleration in all regions in Egyptian seismic map accompanying with damage occurrence of R.C. elements. There is a probability for failure of 6 and 10 stories GLD framed buildings at peak ground accelerations greater than 0.125 g. Therefore, GLD buildings need to be retrofitted to avoid vulnerability.
(4) Seismic performance of EC-94 frames indicates that these buildings behave elastically when exposed to the ground acceleration up to 0.2 g. From 0.2 g to 0.25 g, EC-94 frames may sustain to a slight damage.

Conflict of interest

No conflict of interest.

References

[1] Arnaldo T. Derecho, Seismic design of reinforced concrete structures, in: Farzad Naeim (Ed.), The Seismic Design Handbook, second ed., Van Nostrand Reinhold, USA, 2001, pp. 463–561, Chapter 10.
[2] A. Shuraim, A. Charif, Performance of pushover procedure in evaluating the seismic adequacy of reinforced concrete frames, in: Proceedings of the 7th Saudi Engineering Conference (SEC7), 2007.
[3] P. Fajfar, A nonlinear analysis method for performance based seismic design, Earthquake Spect. 16 (3) (August 2000) 573–592.
[4] Applied Technology Council, ATC-40, Seismic Evaluation and Retrofit of Concrete Buildings, vol. 1, USA, California, 1996.
[5] Nayer A. El-Esnawy, Development of an adaptive pushover procedure for seismic assessment of multistory frame buildings, Al-Azhar Univ. Eng. J., JAUES 2 (3) (April 2007) 241–253.
[6] Farzad Naeim, B. Hussain, M. Roy, Performance based seismic engineering, in: Farzad Naeim (Ed.), The Seismic Design Handbook, second ed., Van Nostrand Reinhold, USA, 2001, pp. 757–792, Chapter 15.
[7] FEMA 273 Federal Emergency Management Agency, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, USA, Washington, D.C., 1997.
[8] FEMA 274 Federal Emergency Management Agency, NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings, USA, Washington, D.C., 1997.
[9] Structural Engineers Association of California (SEAOC), Vision 2000: Performance-Based Seismic Engineering of Buildings, Sacramento, California, 1995.
[10] A.M. Reinhorn, S.K. Kunnath, et al., IDARC2D, A Computer Program for Inelastic Damage Analysis of R.C. Structures. Version 6, USA: Department of Civil Engineering, State University of New York at Buffalo, 2004.

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[11] EC-1994, Egyptian Code for Loads Calculations in Structures and Building Works, Housing & Building Research Center, Egypt, 1994.
[12] ECP-201 2012, Egyptian Code for Loads Calculations in Structures and Building Works, Housing & Building Research Center, Egypt, 2012.
[13] ASCE 31 03, Seismic Evaluation of Existing Buildings, American Society of Civil Engineers, USA, 2003.
[14] FEMA-356, Pre-Standard and Commentary for the Seismic Rehabilitation of Buildings, USA: Washington D.C., 2000.
[15] FEMA 440, Improvement of Nonlinear Static Seismic Analysis Procedures, USA: Washington D.C., 2005.
[16] A. Kadid, R. Chebili, A. Zine, Elasto-plastic analysis of reinforced concrete frame structures, in: Al-Azhar Engineering Ninth International Conference, Egypt, 2007.

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