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Nonlinear Time History Analysis for Seismic Evaluation of 10 and 20-Story Moment Frame Structures Using Performance-Based Design Methodology

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Abstract
The performance-based design (PBD) has been widely recognized as an alternative to the traditional code-based design. The method has been proved to produce more reliable and performance-targeted designs with an advance in using nonlinear modeling and analysis techniques. However, the Egyptian code of practice does not include any guidelines to use the PBD. The current study presents a comprehensive seismic evaluation of two moment-frame reinforced concrete buildings 10 and 20 stories which are representative of the common structural system in Egypt using the performance-based design procedures. The seismic evaluation was performed using two seismic performance levels which are the design-based earthquake (DBE) and maximum considered earthquake (MCE). The nonlinear time-history analyses for both performance levels provided valuable information using transient-inter-story drift, residual drift, and plastic deformation about the over conservatism of the serviceability and strength demands in the Egyptian code.

Keywords: Performance-based design; Seismic evaluation; Egyptian Code of Practice; Nonlinear modeling; Time history; Residual drift; Transient drift; Plastic deformation

1.1 Introduction

Earthquakes are considered one of the most challenging hazards which impact structures specifically high-rise buildings. On 12th October 1992, In Cairo, Egypt had suffered from a major earthquake in 1992 with an epicenter near Dashur with a magnitude of 5 M, Which is considered the most damaging earthquake since 1847. The casualties were 545 deaths, 6512 injured, and about 50,000 homeless people. The personal and financial losses were a motive for a change in the point of view of building code of practice and typical design methods used by engineers who had a prevailing knowledge that Egypt is a seismic –Hazard-free country [1]. Major earthquakes can cause inelastic deformation that leads to local or even global collapse of the structure however current codes are generally based on elastic seismic design. In the current Egyptian code of Loads (ECP-201)[2], the typical procedure to design structures against earthquakes is to perform a gravity analysis then acquire the fundamental period of the structure by an empirical equation which depends on the structural system used and the height of the structure above the base and obtain the design base shear based on a code specified spectral acceleration as ECP-201 has five seismic zones for the whole country ranging from 0.1g to 0.3g and with the base shear equation provided by the code the design base shear can be determined. While it is unpractical to design the structure to behave elastic, so according to most of the current codes the inelastic activity during an earthquake is substituted by dividing the elastic spectral acceleration by a factor R which is named by ECP-201 as the response modification factor and it depends on the structural ductility of the system used. The base shear derived from the code equation is known by the equivalent static lateral load and it is used as a benchmark for the base shear calculated by the response spectrum or dynamic analysis as explained later, after applying the lateral loads on the structures the code specifies an inter-story drift check to be performed to limit the damage of the nonstructural elements so the seismic design within the strength-based design methodology, which adopted my most of the current codes, is an iterative process till the serviceability and strength requirements are satisfied.

1.2 Motivation and Objective

The Egyptian code of practice has two performance limits which are the strength and serviceability limits where all structures designed are checked for both of those limits to be code compliant. While the previous design practice is served the structural profession rather well, the industry advances are pushing the demands for achieving more higher performance goals as an example of the height of the buildings are getting higher due to many reasons as the higher cost of the land, the congestive areas in the city, the need for the prestigious icon of corporates and organizations. Strength-based design is considered the widely used traditional approach for design structures on most of the current codes, it is simple and easy to get strength demand to capacity ratio sufficient to design sections and reinforcement ratio to satisfy the code "ultimate" limit state and perform an inter-story drift check to satisfy the serviceability "damage limitation" state which requires several iterations to satisfy this condition. Many researchers have developed an alternative approach to overcome this iterative issue as Direct-Displacement – based approach which is proposed for buildings initially by *Priestley and Kowalsky* [3]. However, it was still only elastic analysis design is used for design, and inelastic analysis is expressed indirectly through dynamic modification factor as "R" in Eurocode, ASCE-7, and ECP-201. Reinforced concrete is an inelastic material so however elastic analysis is a simple and direct method to get sufficient information about the demand (Moment, shear, and axial forces) but it is still far from accurate and it can result in conservatism or under-estimation of the forces and stresses of the structure. The publication of ASCE 41 [4] was a substantial step towards directly expressing the inelastic analysis in a practical approach and capable of quantifying the deformations of different structure elements (columns, beams, and shear walls). One of the newly developed alternative approaches developed in the last decade is Performance-Based design, there are many definitions of performance-based design, and it can be described as an alternative approach or a methodology for design and achieving multiple objectives that may not satisfy the code restrictions exactly but still compliant with the code performance requirements. The method is recognized by the structural engineering community as the new generation of design and rehabilitation procedures [5][6].

1.3 The objective of the study

The present study is adopting the performance-based design approach as an alternative seismic design method to overcome the issues of the strength-based design in the current codes by presenting three buildings of 10 and 20 story moment frame residential building and 40 stories dual system residential building and all the models designed according to satisfy the Egyptian code of practice (ECP 203-201) and then redesigned using the application of performance-based design. All models are extensively subjected to nonlinear static and time history analyses and the results are discussed in the light of the common engineering demand parameters as the inter-story drift and base shear also the ASCE-41 used to evaluate the results of the plastic hinge rotation thresholds. The importance of this research is to present the application of performance-based design methodology to the seismic design of reinforced concrete structures and demonstrate its capability to achieve the desired performance targets and compare it with the traditional strength-based design approach adopted by the Egyptian code of practice.

1.3.1 Forced-Based design

For many decades, structural engineering practitioners and researchers have been using the forced-based design method to determine the seismic demands which cause straining actions on the structures as the moment, shear, and axial forces. It was also implemented in most seismic codes around the world due to its simplicity to get strength demand to capacity ratios which is considered sufficient to design sections and obtain reinforcement ratios and to satisfy the code ultimate limit state and perform an inter-story drift check to satisfy the serviceability damage limit state. However, this method is more concerned about the strength capacity and overlooks or pays little attention to the deformation capacity. Many researchers have developed an alternative approach to overcome this issue as Direct Displacement–Based approach that was proposed for buildings initially by Priestley and Kowalsky [3]. The main seismic design philosophy has been about using the elastic analysis to incorporate the inelastic response is expressed indirectly through dynamic modification factor as "R" in Eurocode [7], ASCE-7 [8], and ECP-201. Common construction materials such as reinforced concrete and streel are inelastic materials and when subjected to earthquakes, they behave inelastically. Many contemporary building codes are using elastic analysis without explicit use of inelastic modeling methods. Several projects were conducted to address these lacking such as FEMA 356 which developed to be the current version of ASCE 41-13 (Seismic Evaluation and Retrofit of Existing Buildings) [9] was a substantial step towards directly expressing the inelastic analysis in a practical approach and be able to quantify the deformations of different structure elements (columns, beams, and shear walls) [10].

1.3.2 Performance-Based Design

The approach of performance-based design is first introduced to the structural engineering community as a project by FEMA 273[11], SEAOC vision 2000, and ATC-40 [12]. The objective of the three documents to pave the way to an alternative design and assessment method that considers both the strength and deformation and provides guidelines for quantifying the structural performance in terms that are not described in the current codes. While the current codes use forces, displacement, and drifts to quantify the structural behavior, the performance-based design represents the structural performance in new terms such as operational, immediate occupancy, life safety, and collapse prevention. Early researchers recognized the performance-based design as the new generation for the design of and rehabilitation of new and existing structures by using more realistic nonlinear models [6][5].

The development of performance-based design has moved forward since its initial form that presented in FEMA 273 and FEMA 356 to a new version with the publication of the ASCE 41[13] and with more targeted objectives for tall and long-period structures through the publication of tall building initiative [14]

The seismic provisions in the Egyptian code of practice do not permit to implement of the performance-based design and no literature provides guidelines to perform a seismic evaluation for relatively high –rise structures up to 20 stories by using the guidelines of performance-based design whereas this study was motivated to address this research gap.

After performing preliminary analysis and the concrete sections are assumed. The ECP-201 [15] provides two main performance objectives. First, No collapse requirement which states that the structures should be designed to withstand seismic hazard without collapse with an earthquake non-exceedance probability of 10% in 50 years (a return period of 475 years). Second is the damage limitation requirement which states that the structures should withstand seismic demands imposed by earthquakes with a probability of exceedance of 10% in 10 years (return period of 95 years) without excessive damage which is commonly known as a condition to satisfy serviceability performance level. The seismic hazard intensity map in Egypt is divided into six zones. This study was conducted in the most severe intensity zone which is Taba city that located in the northern tip of the Gulf of Aqaba (Zone 5B) with PGA equals to 0.3g.

The ECP-201 provides global acceptance criteria for serviceability in terms of inter-story drifts with no explicit terms of collapse prevention acceptance criteria. The allowable inter-story drift in the ECP-201 is calculated by the following equation (1):

$$d_{rv} / h = 0.7 * R * ds * \nu \leq (0.5\% \text{ to } 1\%) \qquad (1)$$

Where "d_{rv} /h" is the inter-story drift for the serviceability limit state, "R" is the response modification factor which is dependent on the structural system and account for the ductility of the lateral force-resisting structural system and it simply reduces the elastic spectral response demand to a lower limit for cost-efficient inelastic behavior. After performing linear elastic analysis, the drift acquired from the linear analysis using the design spectrum response analysis that represents a design-based earthquake (DBE) of 475 years return period (ds/h) is multiplied by a code-specified factor which is "0.7" to estimate the realistic inelastic drift demand. Several past researchers investigated the over-conservatism of this ratio in the current codes [16] and the ECP-201[17]. While The ECP-201 uses the factor "ν" as a displacement reduction factor that takes into account a lower return period to decrease the drift computed from design based earthquake (DBE) of 475 years return period to service level earthquake (SLE) of 95 years return period that complies with the serviceability limit state code requirement (SLS). The factor "ν" depends on the structure importance factor and varies from 0.4 for importance category (I, II) to 0.5 for category (III, IV)

1.3.3 The nonlinear modeling structural modeling for moment frame elements

The nonlinear modeling and response history analyses for both of the used models were executed using CSI Perform 3D-7 [18]. Instead of using inelastic fiber sections which will increase the computation time substantially, concentrated plastic hinges were assigned for the moment frame column and beam elements to monitor the nonlinear deformations. An IMK lumped plasticity model developed by Ibarra et al. (Ibarra, Medina, and Krawinkler, 2005) was adopted for the study see Figure 1. Haselton et al. developed equations by using experimental results of 255 samples of reinforced concrete rectangular columns to compute the peak and post-peak plastic rotations(Haselton, Liel, and Lange, 2008).

The seismic weight for the nonlinear analyses was taken as DL+0.2 LL. The damping ratio was assumed to be 5% for the elastic design and the nonlinear dynamic analysis for the DBE level while taken as 3 % for the MCE level as recommended by the TBI also Rayleigh damping was taken as 1% of the modal damping.

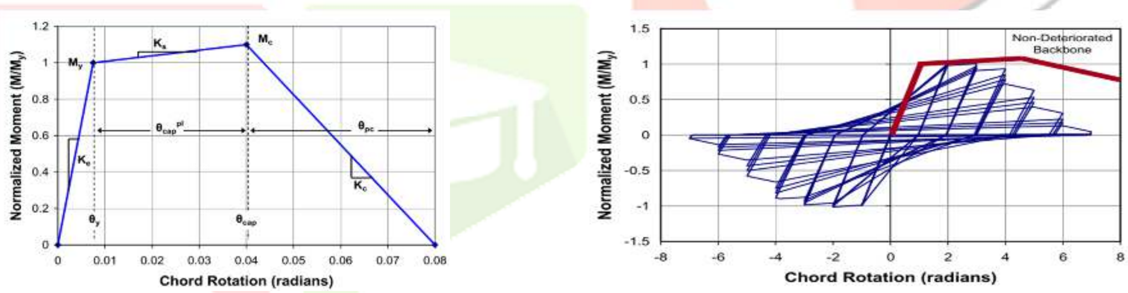


Figure 1

Monotonic and cyclic behavior of the Frame component model used in this study Monotonic and cyclic behavior of component model used in this study. The model developed by Ibarra, Medina, and Krawinkler

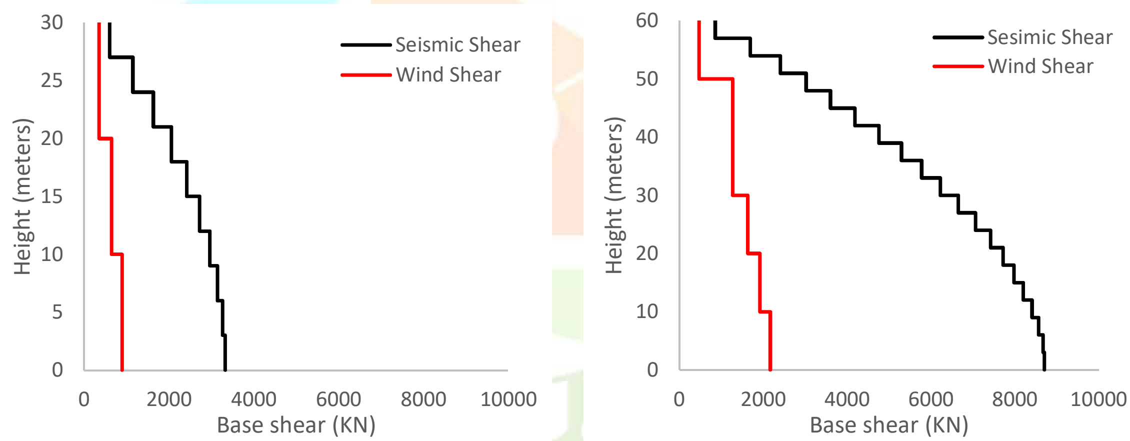
1.4 Prototype structures 10 and 20 story RC moment frame structures

The current study presented two RC moment frame residual buildings with 10 and 20 stories to represent medium to relatively high rise buildings in Egypt with a height of 30 to 60 meters (100 to 195 ft) see Figure 3. The gravity loads on the structure were dead loads that represent the self-weight of all the structural elements, wall and partitions distributed over the slab area 200 Kg/m2, live loads were taken according to the ECP-201 guidelines with residential purposes 250 kg/m2 and floor cover for tiles finishing were assumed to be 150 Kg/m2. A typical structural floor plan with 4 bays in X and Y directions with a span of 6 meters and a typical story height is 3.0 meters. The reinforced concrete grade was 30 Mpa and steel reinforcing was 420/360 Mpa. The slab thickness was taken as 15 cm.

The structures presumed to be located in Taba city with peak ground acceleration of 0.3 g and soil type " B " class and response modification factor of 5 was taken to get the design spectral elastic acceleration curve. The 3D elastic model analysis was performed using CSI ETABS software and modal response spectrum analysis, according to the ECP-201 conditions, was applied with rescaling the value of the response spectrum loads to be 85% of the base shear from the equivalent static load method to satisfy the minimum condition of the ECP-201.Modal analysis results are summarized in Table 1 with first mode periods from dynamic analysis of 1.55 and 2.05 for the 10 and 20 story buildings respectively as seen in Table 1.

All load combinations for gravity loads and lateral loads of earthquake and winds were taken according to the ECP-201 guidelines. A preliminary wind and seismic load analysis were conducted to determine the maximum imposed lateral loads and found that the seismic loads are significantly more critical than the wind loads for both 10 and 20 story buildings as indicated in Figure 2.

All the elements were designed and detailed to be conformed with the strength and serviceability provisions of the ECP-203[21]. As a common practice in Egypt, the thickness of columns was reduced every 3 stories which is a common practice in Egypt as shown in Table 2.

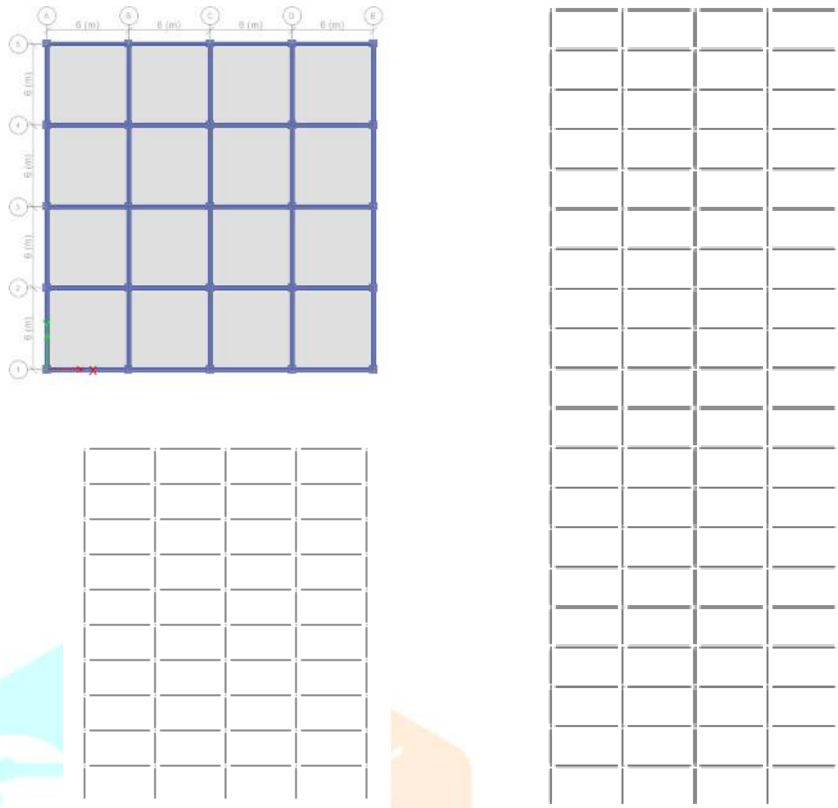


(A) (B)

Figure 2- Wind design loads versus Seismic design loads (A) 10-story, (B) 20-story

Table 1 Modal and response spectrum analysis of 10 and 20 story structures

Building	Code Fundamental Period	Mode Period			Base shear (KN)			Response Spectrum Scaling Factor
		1st Mode	2nd Mode	3rd Mode	Equivalent Static Load Method (V)	85 % V	Response Spectrum	
10-Story	0.96	1.55	1.34	0.5	3836	3261	3047	1.07
20-Story	1.62	2.05	1.64	0.70	10271	8730	7936	1.1



(B) (C)

Figure 3 (A) Floor Plan Layout of 10 and 20 Story Building -2D Perform Model of 10-Stor 20-Story (B) 10-story, (C) 20-story

1.5 Project site and Response spectrum curves (Design Level and Maximum considered Earthquake)

The chosen location for all the current case studies is Taba city in Egypt. It represents a tourism attractive city on the red sea with a high potential for construction investment. The city is located in the highest seismicity zone by the Egyptian code of loads (Zone 5B) which corresponds to a peak ground acceleration of 0.3g. The soil condition is assumed to be dense soil of sand and gravel which corresponds to the "B" class and importance factor of a category (I). The study used the ECP-201 design spectral response spectrum (DBE) that corresponds to earthquake hazard with a return period of 475 years for the design of both 10 and 20 story buildings and used another earthquake hazard that corresponds to the maximum considered earthquake (MCE) level with much rarer occurrence. Due to the lacking of the guidelines in the ECP-201 for taking such rarer seismic hazard, the study took the same concept of the ASCE-07 that relates the DBE level to 2/3 of the MCE level. It is worth noting that the ASCE -7 code relates the MCE level to a 2% probability of exceedance in 50 years that corresponds to a return period of 2475 years. Besides, the study used a higher importance factor of 1.2 to be associated with the MCE level by considering category II of the ECP-201. The DBE and MCE response spectrum curves are presented in Figure 4.

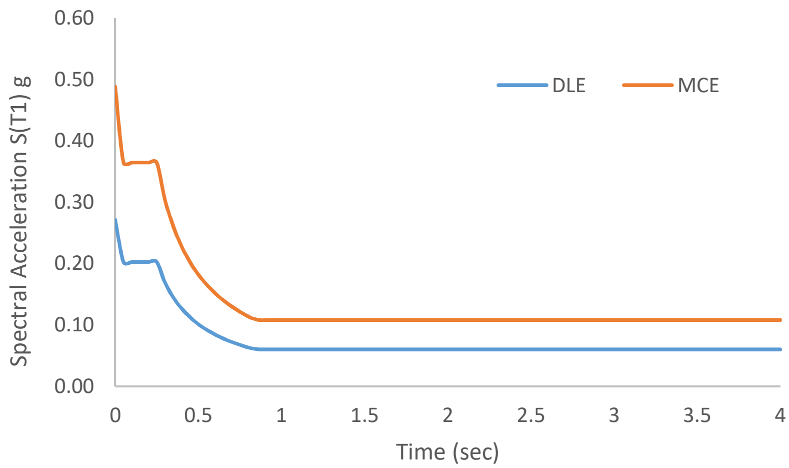


Figure 4- DBE and MCE response spectrum curves.

Table 2- Prototype structures design as per the ECP

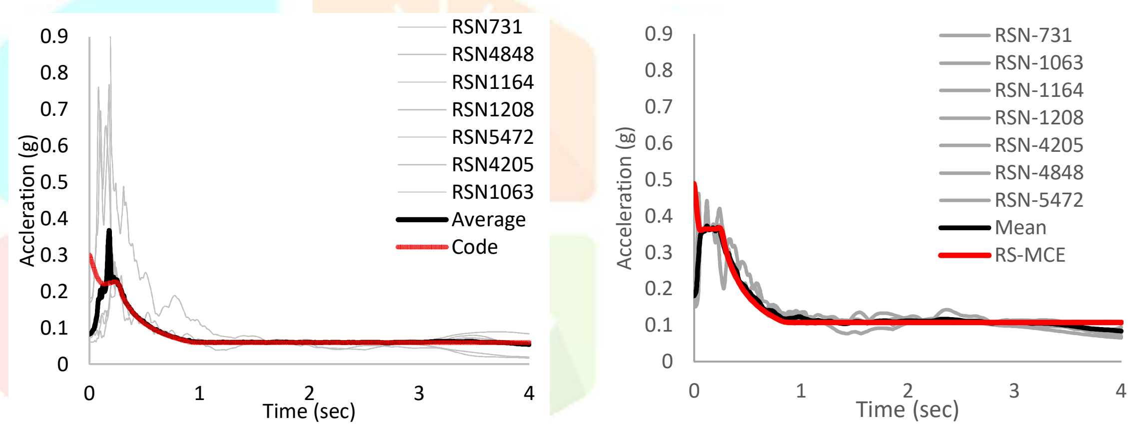
Building	Layout	Section Details			
		Story No.	Column sections (mm)	Column RFT	Beam (mm)
10-Story Moment Frame Building	Interior frame	1-2	800x800	28Ø20	300x750 5 Ø16 Top &4 Ø16 Bottom
		3-4	700x700	24Ø20	
		5-7	600x600	20Ø20	
		8-10	600x600	16Ø20	
	Exterior frame	1-3	700x700	28Ø18	300 x 750 5 Ø16 Top 4 Ø16 Bottom
		4-10	600x600	20Ø18	
20-Story Moment Frame Building	Interior frame	1-5	1150x1150	48Ø22	550 x 800 (Story 1-20) 6 Ø20 Top 5Ø20 Bottom
		6-10	1050x1050	40Ø22	
		11-15	950x950	32Ø22	
		16-20	850x850	28Ø22	
	Exterior frame	1-5	1150x1150	48Ø22	500 x 900 (Story 1-10) 6 Ø20 Top 5Ø20 Bottom
		6-10	1050x1050	40Ø22	
		11-15	950x950	32Ø22	500x800 (Story 11-20) 6 Ø20 Top 5Ø20 Bottom
		16-20	850x850	28Ø22	

1.6 Ground motions selection and spectral matching

One suite of seven pairs of ground motions with two orthogonal directions were used for the study of the 10,20 story buildings see Table 3. All ground motions were properly matched to the ECP code-specified design elastic response spectrum (DBE) and the maximum considered earthquake (MCE) level for collapse prevention assessment as in Figure 5. The earthquake data was acquired from the PEER database (The Pacific Engineering Earthquake Center), with a magnitude ranging from 6.6 to 7.5 M on the Richter scale with periods of interest 0.191s, 0.32 s, 1.44 s, 2.44 s. The ground motions were selected to the design response spectrum using the tool provided by the PEER website. The selected ground motions satisfy the ASCE 7-16 condition that the average of the SRSS spectral from all horizontal component pairs does not fall below the design response spectrum in the period range of interest 0.2T to 1.5T, where T is the fundamental period of the building. The spectral matching approach developed by (32) was performed to match the original ground motions with the ECP-code design response spectrum by using SeismMatch software. SeismoMatch is an application capable of adjusting earthquake accelerograms to match a specific target response spectrum, using the wavelets algorithm proposed by Abrahamson [1992] and Hancock et al. [2006] or the algorithm proposed by Al Atik and Abrahamson [2010]. All the spectrally matched ground motions are found to be in good convergence with the original time histories.

Table 3- Ground motions data

No	Record Number	Sequence	Earthquake Name	Station Name	Magnitude	Year
			Used for Case Study 1, 2			
1	731		Loma Prieta	"APEEL 10 - Skyline"	6.93	1989
2	1208		Chi-Chi Taiwan	"CHY046"	7.62	1999
3	4205		Niigata-Japan	"NIG015"	6.63	2004
4	4848		Chuetsu-oki_Japan	"Joetsu Ogataku"	6.8	2007
5	5472		Iwate_Japan	"AKT017"	6.9	2008
6	1063		Northridge	Rendeli	6.69	1994
7	1164		Kocaeli_ Turkey	"Istanbul"	7.51	1999



(A) **(B)**

Figure 5- Spectral matching of ground motions (A) DBE level - (B) MCE level

1.7 Results and discussion

The results of nonlinear time history analyses (NTHA) are presented in terms of mean value and standard deviation of some important engineering demand parameters (EDP) as the peak transient inter-story drift at the DBE and MCE levels, the residual inter-story drift at the MCE level, and chord plastic rotation at the most critical earthquake of the MCE suite. All are considered significant and valuable parameters in the framework of performance-based design to produce a reliable damage and risk assessment analysis.

The mean values and standard deviation of the peak transient inter-story drift at each floor subjected to the matched seven ground motions of both the design-based earthquake level (DBE) and the maximum considered earthquake level (MCE) were presented as shown in Figure 6. As shown in Figure 6 (A, B) the light black line represents the ECP-201 allowable drift threshold for the design-based earthquake of the return period of 475 years without using the reduction factor that emulates the lower return (95 years) period for serviceability requirement that previously discussed which presented in the dark black line. The mean peak inter-story drift values from NTHA for a suite of seven matched ground motions with found not exceeding 0.27% and 0.3% for the 10 and 20 stories respectively show the over-conservatism of the inter-story drift requirements of the ECP-201 as the maximum lower and upper bounds corresponds to 0.5 % and 1% that both buildings 10 and 20 stories were proportioned and designed to withstand. Even with the increasing seismic demand to the MCE level and increasing the important factor for the building, still, the peak inter-story drifts were 0.3% and 0.35% for the 10 stories and 20 stories respectively. It is worth mentioning that these values are satisfying the immediate occupancy condition according to the tall building initiative guidelines (TBI) that takes maximum story drift for serviceability to 0.5%.

The residual inter-story drift or the permeant deformation is the story drift ratio at a location in a structure at rest following response to earthquake motion [14]. It is considered an important parameter when it comes to identifying the degree of sustained damage of the structure after the seismic event and is widely used in the damage assessment within the framework of performance-based design. Yet, it is not a requirement by the ECP-201. The current study used, for the collapse assessment, the TBI limits of the residual drifts that correspond to the mean value of 1% for a suite of seven ground motions. As it can be noticed in *Figure 7*, the mean value of the seven MCE ground motions was not exceeding 0.1% which is about 10% for the collapse prevention acceptance criteria.

As for the deformation-controlled elements, as previously explained the study used a concentrated plastic rotation for all of the moment-frame column-beam elements to monitor the plastic deformation in these elements. The results of NTHA revealed that the IWATE-earthquake is the most critical ground motion that gives the maximum deformations. And the beams are the most affected elements as shown in *Figure 8*, the profile for the chord plastic rotation in both of the beam ends is presented as H1 for the left beam-end plastic hinge and H2 for the right beam-end plastic hinge. Both exterior and interior beams were for the 10 and 20 story buildings respectively were found to be substantially below the ASCE 41 collapse prevention (CP) limit of 4%.

1.8 Conclusion

The current study presented a demonstration for the procedures of performance-based design methodology using state of the art nonlinear structural modeling and time history analyses techniques to provide a reliable structural analysis for two-moment frame 10 and 20 story buildings up to 60 meters height. The nonlinear dynamic analyses presented in the study comprehensive analyses on the global performance of the structures represented in the transient and the residual inter-story drifts to simulate and give the best estimate of the damage sustained during and after an earthquake shaking. Besides insights over the local structural performance represented in the deformation of the moment frame column-beam elements. The results of the inter-story drifts were compared against the TBI limits and the plastic deformation results were evaluated with the ASCE 41 acceptance criteria and all found to be below the limits with significant margins that could be used to decrease the concert size of the elements and the reinforcement ratio to provide more cost-effective design and also enhance the structural performance and enforce more ductile behavior.

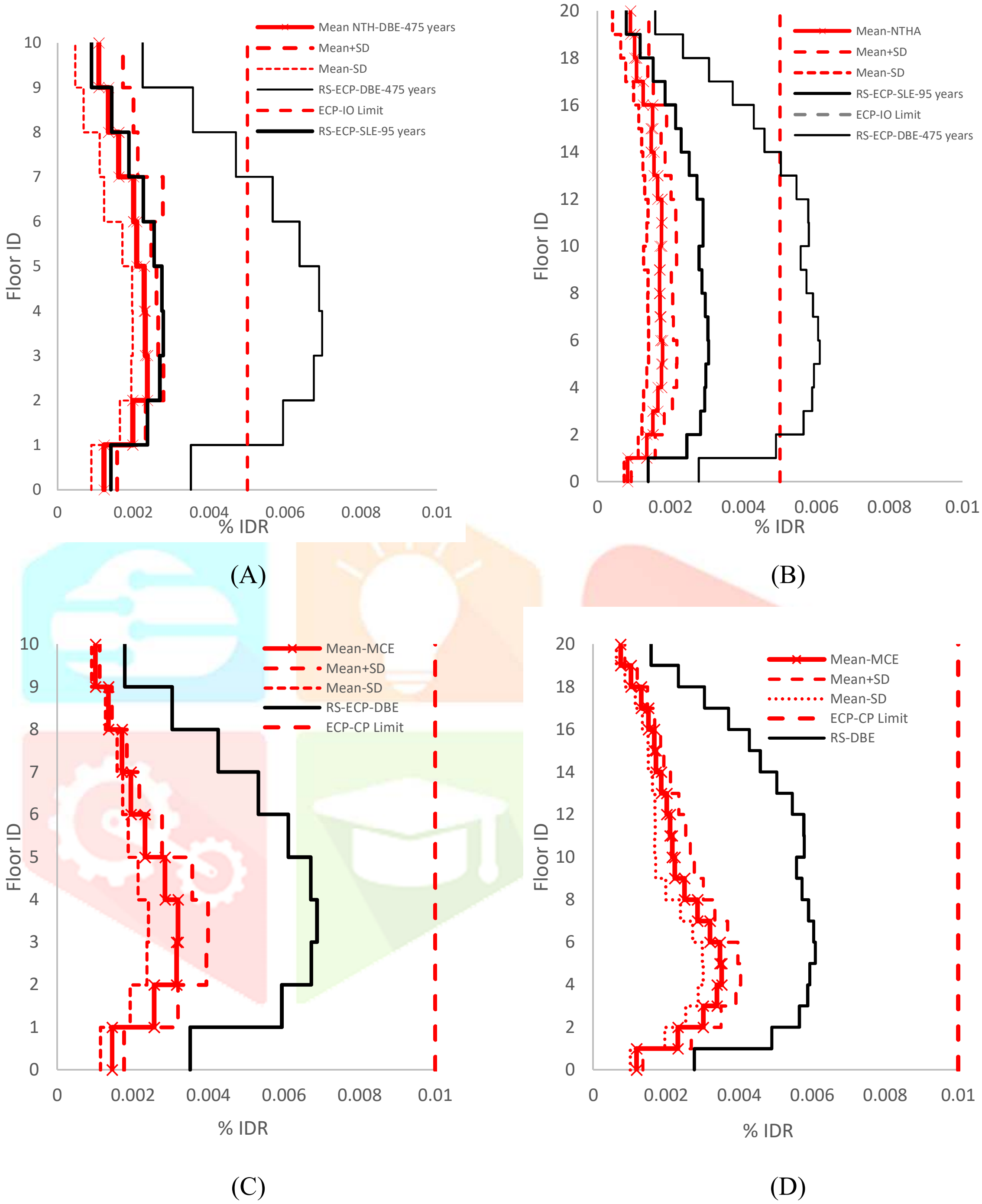
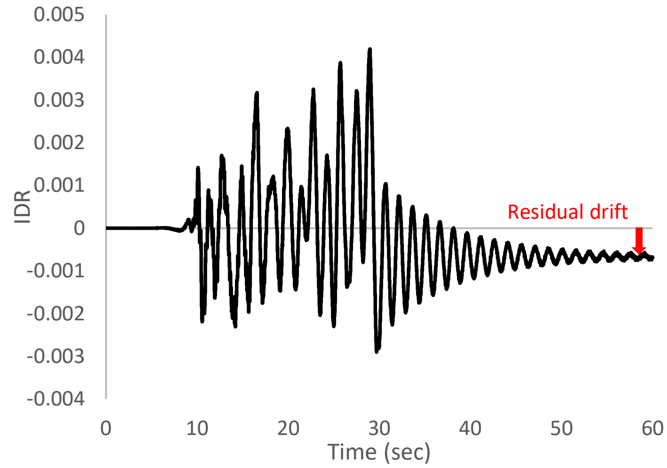
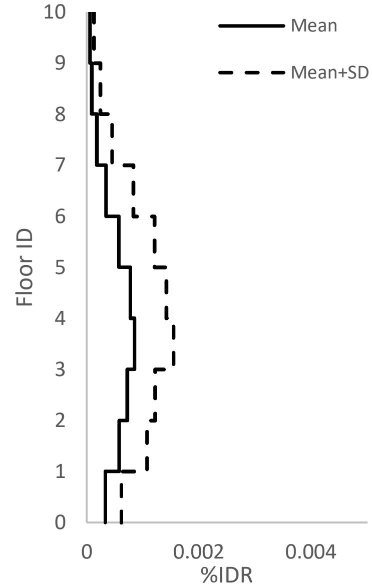
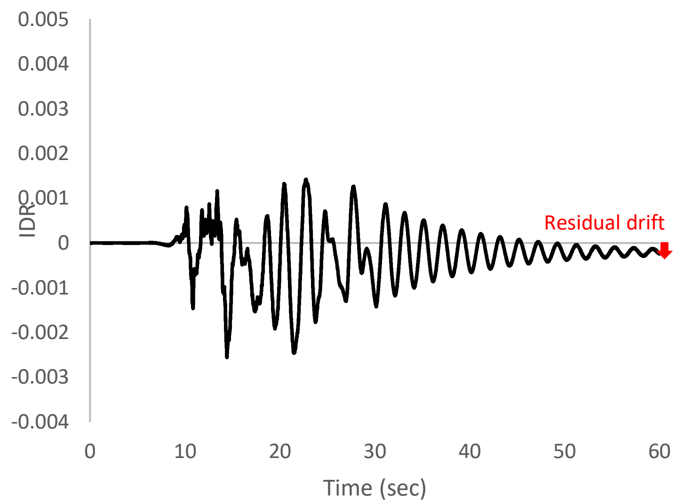


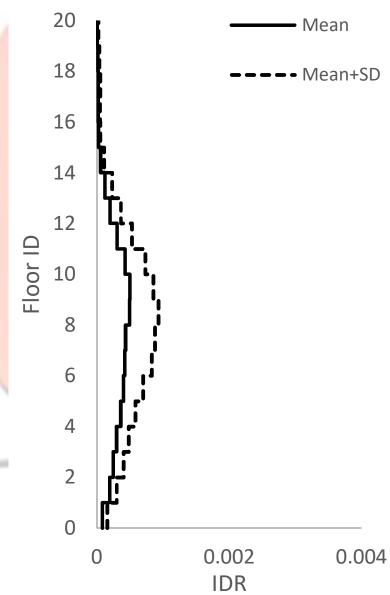
Figure 6- Peak transient inter-story drifts (A) DBE-10 story (B) DBE-20 story (C) MCE-10 story (D) MCE 20-story





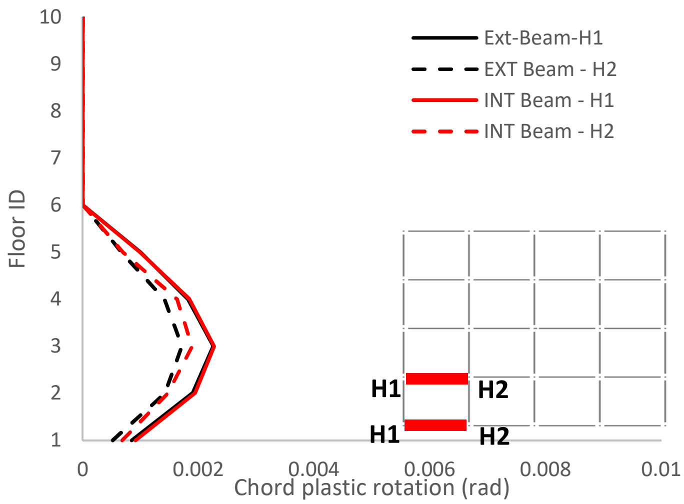
(A) (B)



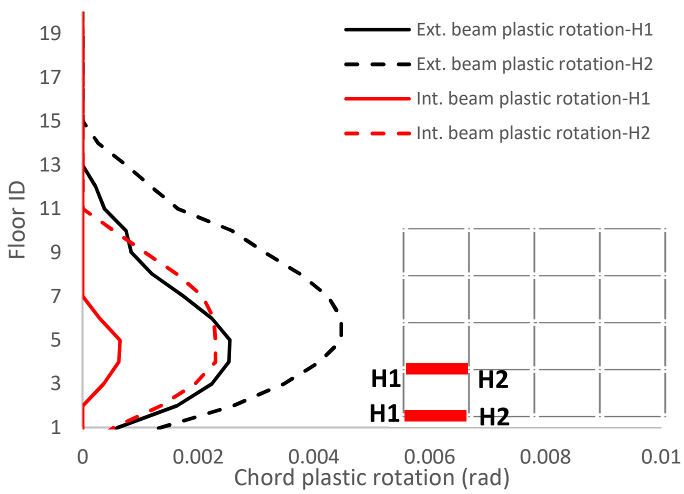


(C) (D)

Figure 7- Mean residual drifts for the MCE level of seven matched earthquakes. (A) 10-story-Cheutsu earthquake drift time history (B) mean residual drift profile 10-story (C) 20-story-Cheutsu earthquake drift time history (B) mean residual drift profile 20-story



(A)



(B)

Figure 8- Plastic rotation of Exterior and Interior beams in both beam-ends (left hinge-H1)(right hinge-H2, (A) 10-story, (B) 20-story

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