IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 15, Issue 1 Ver. III (Jan. - Feb. 2018), PP 66-86
www.iosrjournals.org

Evaluation of Seismic Response Modification Factor of Multistory Buildings DesignedAccording to Egyptian Code

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Abstract: *This study is mainly concerned with the calculation of the values of response modification factor at failure for idealized reinforced concrete moment resisting multistory frame systems designed according to the Egyptian code of loads ECP-201 (2012). Parametric studies are carried out for RC moment resisting frame with 3, 6 and 9 stories that are modelled in three-dimensions as residential buildings with different configurations and variable parameters. SAP2000 software is used to model and analyze these systems using three-dimensional nonlinear static pushover analysis considering material and geometrical nonlinearity. The buildings are studied under the effect of several parameters such as single or multi-bay frame, number of stories, seismic zone intensity and type of spectrum according to Egyptian code. Their effect on pushover curve, R-factor and its components are analyzed. Comparisons between the results show the difference of some values and the indifference of others values including R-factor values.*
Keywords: *Base shear, Pushover analysis, Response modification factor, Seismic Zones, Spectrum type.*

Date of Submission: 26-01-2018 Date of acceptance: 15-02-2018

I. Introduction

The seismicity of Egypt is characterized by small to moderate earthquake activities due to the relative motions between the African, Arabian and Eurasian plates. The highest seismicity rates are found at the eastern boundaries of Egypt, viz. the Gulf of Aqaba, which forms the southern end of the Dead Sea Fault, and the northern part of the Red Sea. A revised earthquake catalogue for Egypt and its surroundings during the period from 2200 BC to 2009 AD with magnitude equal or greater than three is compiled using information from several international and local seismic catalogues, figure 1, (Abuo El-Ela et al, 2012) [1]. According to ECP-201 [2012] [2], Egypt is divided into five seismic zones according to the value of design ground acceleration, figure 2.

ECP-201 [2012] quoted the Euro-code design response spectra EC8 [2004] [3] which is based on expected surface-wave magnitude (Ms) at site. Two different spectra, based on expected surface-wave magnitude (Ms) at site, namely in EC8 Type (I) and Type (II) are renamed in ECP-201 as type (2) and type (1). ECP-201 specify for coastal zones on the Mediterranean Sea (40 km distance from shore), the both response spectrum curves, type (1) and type (2). For all other zones throughout Egypt (which include many regions with expected surface-wave magnitude Ms > 5.5), the response spectrum curve type (1) is specified.

The basic principal of designing structures for strong ground motion is that the structure should not collapse but damage to the structural elements is permitted. Since a structure is allowed to be damaged in case of severe shaking, the structure should be designed for seismic forces much less than what is expected under strong shaking, if the structures were to remain linearly elastic. Response reduction factor is the factor by which the actual base shear force should be reduced, to obtain the design lateral force. ECP-201 [2012] gives a value of '*R*' equal to 5.0 to 7.0 for moment resisting frames with limited and sufficient ductility respectively.

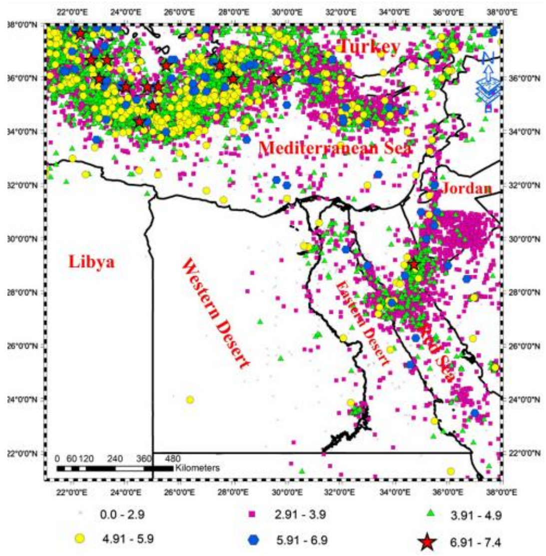


Fig. 1: Local and regional seismic activity Seismicity of Egypt based on the compiled earthquake catalogue

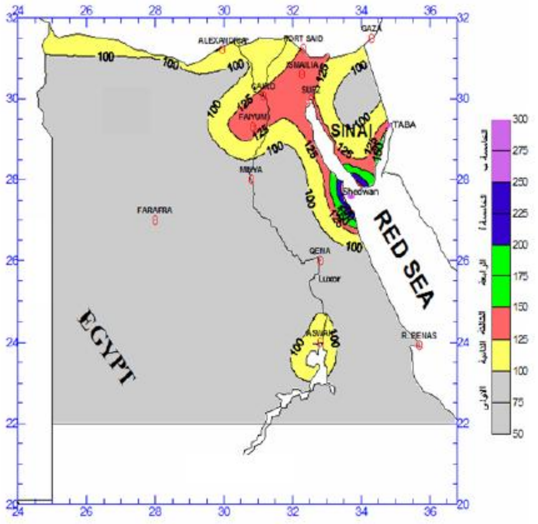


Fig. 2: Seismic Zones in Egypt, probability of exceedance of 10 % in 50 years (return period of 475 years), ECP-201 (2012)

Several authors have worked on reinforcement concrete moment resisting frame building systems focusing on the performance level of the systems with low, intermediate and high ductility and study of seismic behaviour and response modification factor of the systems [Jain and Navin,(1995) [4], Borzi and Elnashai (2000) [5], Elnashai et al (2002) [6], Lee et al. (2005) [7], Zafar (2009) [8], Zahid et al (2013) [9], Chaulagain et al (2014) [10], Al-Ahmar and Al-Samara (2015) [11]]. The criteria used for the evaluation of the R-factor for moment resisting frame structure through its components is the nonlinear static pushover analysis and incremental dynamic analysis (Nonlinear Time History Analysis) using both material and geometrical nonlinearity [Vamvatslos and Cornell (2001) [12], Massumi, et al (2004) [13]].

The main goal of the research conducted for this study is determining a reasonable value for the seismic response modification factor, R, for multi-story one-bay frames and multi-story multi-bay frames when designed using the Equivalent Lateral Force Procedure (ELFP) of the ECP-201 (2012) [spectrum type 1 and 2]. A secondary goal of this research is evaluating the percent of base shear for Multi-bay frames calculated according to ECP-201 (2012) (spectrum, type 1 and 2) and IBC (2012) [14]. To achieve these goals, the reinforcement concrete building is modelled, loaded, and designed according to ECP-201 (2012). After that, a pushover analysis is performed by subjecting the structure to a monotonically increasing pattern of lateral seismic loads representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially and the pushover curve representing force displacement relationship until failure can be plotted for each frame structure to obtain the R-factor.

In this paper, RC limited ductility framed buildings with 3, 6 and 9 stories have been designed according to ECP-203 (2007) [15] for two seismic zone intensity 0.15g and 0.25g using ECP-201 (2012) (spectrum type 1 and 2). The empirical equation of fundamental period of vibration (T) given by the code and the accurate value calculated by SAP2000 program have been calculated. Design base shear for two seismic zone intensity 0.15g and 0.25g using ECP-201 (2012) [spectrum type 1 and 2] as well as IBC (2012) have been determined and compared. Nonlinear pushover static analysis has been performed to determine hinge status and corresponding base shear at yield and ultimate states. The values of response modification factor R for RC limited ductility framed buildings are evaluated in two seismic zone intensities 0.15g and 0.25g using the both types of design response spectra. The resultant values of response modification factor are compared with these values given in ECP-201 (2012). The results are discussed and recommendations are given in this field.

II. Concept For Determining Response Modification Factor

Most of the codes used for the seismic design of buildings use single factor to reduce the forces caused by earthquakes. This factor, which have different value in various codes for the same types of structures, is called response modification factor (R-factor) in the Egyptian code ECP-201 (2012), behavior factor in Euro code EC8 (2004) [3], and response modification coefficient in ASCE (2010) [16]. The factor accounts for the nonlinear response of a structure by taking advantage of the fact that the structures possess significant reserve strength and capacity to dissipate energy, called over strength and ductility, respectively, [ATC, (1995a) [17]; Borzi and Elnashai (2000) [5], Asgarian and Shokrgozar (2009) [18]].

Accordingly, the structure is designed for much less base shear forces than would be required if the building is remained elastic during severe shaking at a site. Such large reductions are mainly due to two factors: the ductility reduction factor (Rμ), which reduces the elastic demand force to the level of the maximum yield strength of the structure, and the over-strength factor, (Ω), which accounts for the over-strength introduced in code-designed structures. Thus, the response reduction factor (R) is:

$$R = R\mu \times \Omega \qquad (1)$$

The relation between the base-shear of a structure and its roof displacement which can be calculated by a nonlinear static analysis has been illustrated in figure 3.

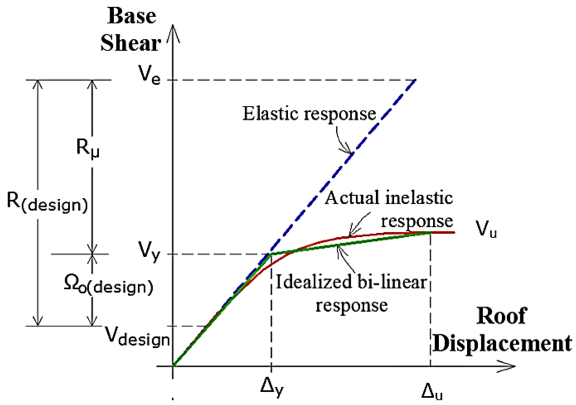


Fig. 3: Relationship between force reduction factor (R), structural over-strength (Ω), and ductility reduction factor (Rμ)

2.1 Over-strength factor Ω

The over-strength factor (Ω) can be defined as the ratio of the actual to design level strength (Elnashai and Mwafy, 2002 [19]). It can be expressed as:

$$\Omega = V_u / V_d \qquad (2)$$

where V_u is the actual strength and V_d is the design strength

The main sources of the structural over-strength results from sequential yielding of critical regions, material over-strength, strain hardening, capacity reduction factors, member size, nonstructural elements and special ductile detailing (Elnashai and Mwafy, (2002) [19]; Freeman, (1990) [20]; Lee *et al*., (2005) [7]; Rodrigues *et al*., (2012) [21], Varum, (2003) [22]).

2.2 Ductility reduction factor, Rμ

The extent of inelastic deformation experienced by the structural system subjected to a given ground motion or a lateral loading is given by the displacement ductility ratio 'μ' (FEMA-451, (1999) [23]). The inelastic behaviors of a structure can be idealized as:

$$\mu = \Delta_u / \Delta_y \qquad (3)$$

where μ is the displacement ductility ratio, Δ_u is the ultimate displacement and Δ_y is the yield displacement. Yield displacement and yield base shear are judged through an idealization of the capacity curve.

Ductility reduction factor Rμ is a function of structural characteristics such as ductility, damping and fundamental period of vibration (T), and the characteristics of earthquake ground motion (Maheri and Akbari, (2003) [24]). Researchers proposed different formulations in order to determine the ductility reduction factor Rμ, (Newmark and Hall, (1973) [25]; Uang (1991) [26], Paulay and Priestly, (1992) [27], Miranda and Bertero, (1994) [28]; Kappos (1997) [29], Priestley, (2000) [30]; Elnashai and Mwafy (2002) [21], Mondal et al (2013) [31]).

In this study, the formulation proposed by Paulay and Priestley (1992) [27] is used.

Rμ = 1.0 for zero-period structures

$R\mu = \sqrt{2\mu - 1}$ for short-period structure

$R\mu = \mu$ for long-period structure

$R\mu = 1 + (\mu-1)\, T/0.70 \quad (0.70 < T < 0.30)$ (4)

where $R\mu$ is the ductility reduction factor and μ is the displacement ductility.

2.3 Provisions of '*R*' factor in international codes and guidelines

The response reduction factor in different codes and guidelines varies depending on the type of structural system and ductility class of the structures. For RC frames, values of '*R*' as specified in, UBC 97 [32], IBC 2012, Eurocode-8, ECP 2012 are presented in Table 1.

UBC 97and IBC 2012 divided RC buildings into three ductility classes. The values are within the range of 3.0 to 8.0 for ordinary. Eurocode-8 gives the behavior factor for regular RC frame structures for two ductility classes. Eurocode-8 (2004) specified the over-strength factor (the ratio of V_u/V_y) as 1.30 in multi-story multi-bay frames. ECP (2012) gives a value of '*R*' equal to 5.0 to 7.0 for moment resisting frames with limited and sufficient ductility respectively.

Table 1: R values allocated in different codes for concrete frame structures

Structural system	R value			
	UBC97	IBC 2012 ASCE7-10	Eurocode-8	ECP 2012
Medium ductility class (DCM) High ductility class (DCH)			3.0 V_u/V_y 4.5 V_u/V_y	
Limited ductility frame Sufficient ductility frame				5.0 7.0
Ordinary moment frame Intermediate moment frame Special moment frame	3.5 5.5 8.5	3.0 5.0 8.0		

For multi-bay multi-story $V_u/V_y = 1.3$, and for single-bay multi-story $V_u/V_y = 1.2$

2.4 Seismic base shear and design response spectra in international codes

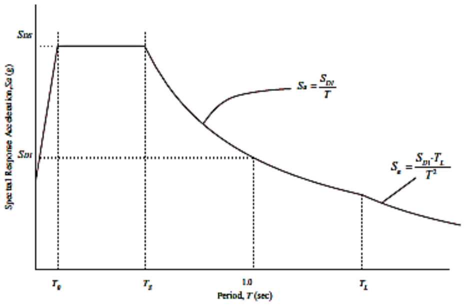
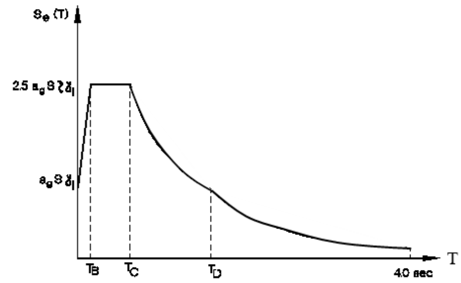
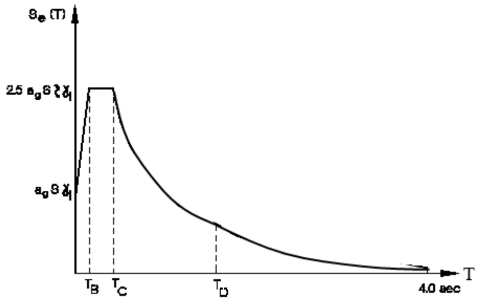
Design spectrum depends on level of ground motion expected at site and local sub-soil. Codes specify standard spectral shapes which are scaled for PGA or other spectral ordinates and amplification factors corresponding to site classes. Table 2 gives summary about Seismic base shear and design response spectra in some international codes.

ASCE 7 (2010) [16] considers the amplification effect more rationally by specifying amplification factors depending on amplitude of spectral ordinates.

EC8 (2004) specify amplification factors for various soil types, independent of ground shaking levels. To consider ground shaking levels two different spectra, based on expected surface-wave magnitude (*Ms*) at site, namely Type I and Type II are specified. According to EC8 (2004), it states that: "*If deep geology is not accounted for, the recommended choice is the use of two types of spectra: Type I and Type II. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, Ms, not greater than 5,5, it is recommended that the Type I spectrum is adopted. Different spectra may be defined in the National Annex, if deep geology is accounted"*

ECP-201 (2012) specify for coastal zones on the Mediterranean Sea (40 km distance from shore), the both response spectrum curves, type (1) and type (2). However, for all other zones throughout Egypt (which include many regions with expected surface-wave magnitude *Ms* > 5.5) the response spectrum curve type (1) [Type (II), EC8] is specified.

Table 2: Summary of Seismic base shear and design response spectra in some international codes

Action	IBC 2012 –ASCE7-10	EC8 (2004) - ECP-201(2012)
Base Shear	V = Cs W Cs = The seismic response coefficient, equal S_{DS}/ (R/I) SDS= the design spectral response acceleration parameter in the short period range R = the response modification factor I = the importance factor W= Total weight of the building. The value of *Cs* need not exceed the following: Cs=S_{DI} / [T(R/I)] for T> T_L Cs=S_{DI} / [T_L(R/I)] for T> T_L *SD*1= the design spectral response acceleration parameter at a period of 1.0 s, T = the fundamental period of the structure T_L = long-period transition period	V= Sd(T) λ W/g S_d(T)= Ordinate of the horizontal design spectrum [type (I) or type (II)] for elastic structural analysis at period T λ = Correction factor W = Total weight of the building
Design Response Spectrum	 $$S_a = \begin{cases} S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) & \text{for } T < T_0 \\ S_{DS} & \text{for } T_0 \leqq T \leqq T_S \\ \frac{S_{D1}}{T} & \text{for } T_S < T \leqq T_L \\ \frac{S_{DS}\,T_L}{T^2} & \text{for } T > T_L \end{cases}$$ Where T_0=0.2 S_{DI}/S_{DS} T_S=S_{D1}/S_{DS} S_{DS}=2/3 S_{MS}= 2/3 F_a S_s S_{DI}=2/3 S_{MI}= 2/3 F_v S_l S_{Ds} = 2.75 a_g S_{DI} = 1.25 a_g F_a= site coefficient ranges from 0.8 to 2.5 for site classes A, B, C, D and E F_v = site coefficient ranges from 0.8 to 3.5 for site classes A, B, C, D and E	 Type (I) [EC8] Type 2 [ECP (2012)]  Type (II) [EC8] Type 1 [ECP (2012)] $0 \leq T \leq T_B$ $$S_d(T) = a_g\ \gamma_I\ S \left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{R} - \frac{2}{3}\right)\right]$$ $T_B \leq T \leq T_C$ $$S_d(T) = a_g\ \gamma_I\ S\ \frac{2.5}{R}$$ $T_C \leq T \leq T_D$ $$S_d(T) = a_g \gamma_I\ S\ \frac{2.5}{R}\left[\frac{T_C}{T}\right] \geq [\,0.20\,]\ a_g\gamma_I$$ $T_D \leq T \leq 4$ sec $$S_d(T) = a_g\ \gamma_I\ S\ \frac{2.5}{R}\left[\frac{T_C\ T_D}{T^2}\right] \geq [\,0.20\,]\ a_g\gamma_I$$ a_g=Design ground acceleration γ_I = Importance factor R=Response modification factor

III. Nonlinear Static Pushover Analysis Method

3.1 Purpose of pushover analysis

The purpose of pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands in designing earthquake resistant buildings by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element and connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behavior. The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis, (Krawinkler et al (1998) [33]).

A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads, various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic nonlinear force displacement relationship can be determined.

3.2 Plastic Deformation Curve

Performance-based engineering yields structures with predictable performance within defined levels of risk and reliability (SEAOC Vision 2000 [34], FEMA 356 [35] and ATC 40 [36]). The critical outcome is the prevention of total structural collapse. This means that the upper level withstands total collapse (CP); the sub level, for the crucial structures, may be slightly damaged but remains fit for immediate occupancy (IO). Between the sub and upper levels there is Life Safety (LS) level situation. The nonlinear procedures of FEMA require definition of the nonlinear load deformation relation. Such a curve is given in figure 4.

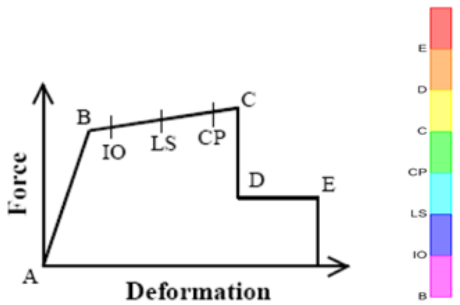


Fig. 4: Typical load – deformation relation and target performance levels

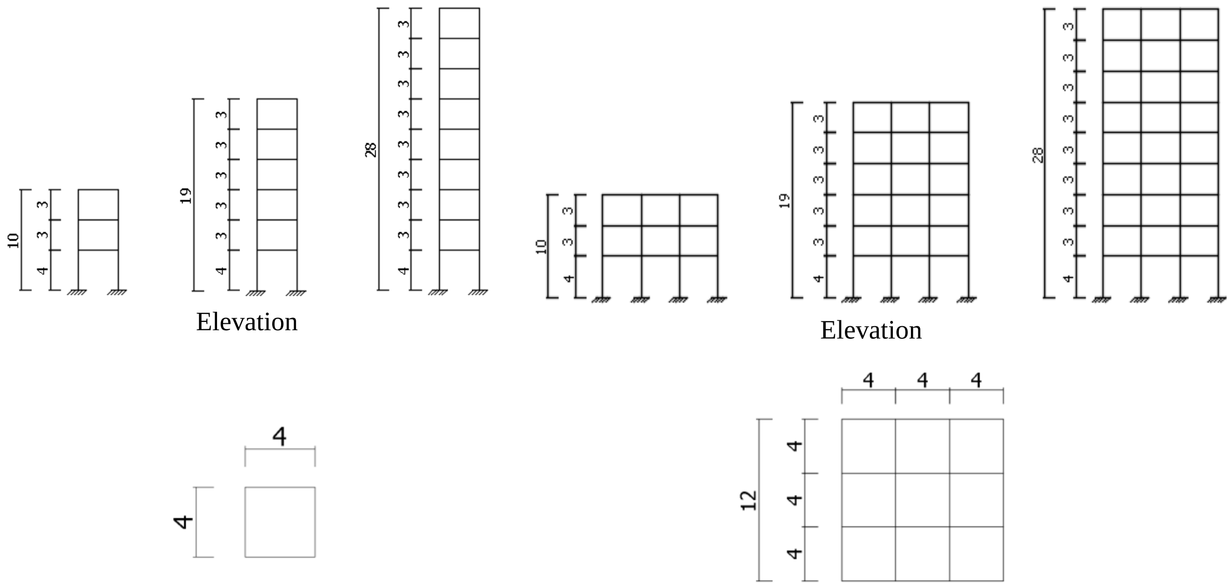
The five points (A, B, C, D and E) are used to define the hinge rotation behaviour of RC members according to FEMA. Three more points Immediate Occupancy (IO), Life Safety (LS) and (Collapse Prevention) CP, are used to define the acceptance criteria for the hinge. Multiple performance objectives for these levels, including the seismic transformation periods, have been specified in Table 3.

Table 3: Required seismic performance levels for design earthquakes (EQ)

Purpose of structure and class of buildings	Exceeding probability of EQ		
	50 years 50%	50 years 10%	50 years 2%
	Average return period		
	75 year	475 year	2500 year
Buildings to be utilized after the EQ	-	IO	LS
Intensively and long-term occupied buildings	-	IO	LS
Intensively and short-term occupied buildings	IO	LS	-
Buildings containing hazardous materials	-	IO	CP
Other buildings		LS	-

IV. Discription Of Buildings Model Designed According To Egyptian Code

RC multi-story three bays frame buildings with 3, 6 and 9 stories have been analysed using SAP2000 structural analysis software package (2016). The buildings are modelled as 3D frame structure using frame elements for columns, longitudinal beams and transverse beams and shell element for slabs with rigid floor diaphragms distribute uniformly the lateral loads on the vertical elements. Figure 3 shows elevation and plane layout for buildings dimensions. Material properties for reinforced concrete buildings are illustrated in table 4. Stress-strain curves for concrete and, steel bars are illustrated in figure 4.

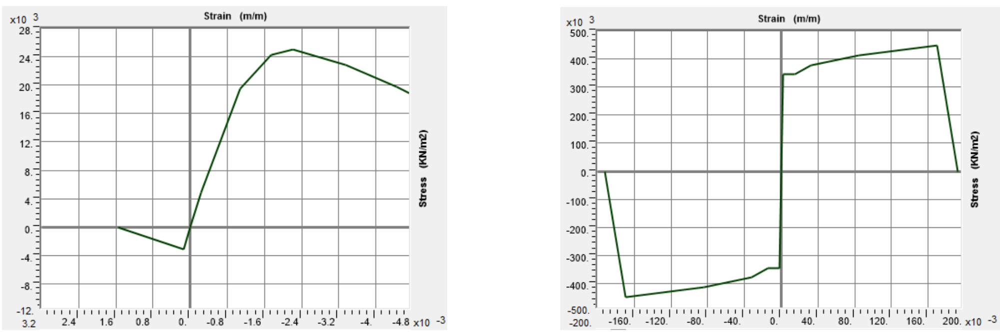


(a) Single-bay frames **(b) Multi-bay frames**

Fig. 3: Layout of studied buildings

Table 4: Material Properties for Buildings

F'c	25000 kN/m²	concrete strength
Fy	345700 kN/m²	rebar yield strength
Ec	22000000 kN/m²	modulus of elasticity of concrete
Es	2.0E+8 kN/m²	modulus of elasticity of rebar
G	10356491 kN/m²	Shear modulus
Y	0.2	Poisson's ratio



(a) Stress-strain curve for concrete (b) Stress-strain curve for steel bar

Fig. 4: Stress-strain curves introduced in SAP2000 (Computer & Structures Inc., 2011)

The following loading assumptions have been considered:
1) Total Dead Load (D) is equal to DL+SDL+CL
2) Dead Load (DL) is equal to the self-weight of the members and slabs.
3) Super-imposed Dead Load (SDL) equals to 3.0 kN/m². SDL includes partitions and ceiling weight.
4) Cladding Load (CL) is applied only on perimeter beams.
5) Live Load (L) equals to 2.0 kN/m².

The studied buildings are subjected to different types of load combinations according to ECP 2012. These combinations are applied by the following terms:

U = 1.40 D + 1.60 L
U = 1.12 D + α L ± S (5)
Where D is the dead load, L is the live load; S is the seismic load and superposition factor of the structure's the residential buildings.

RC frame buildings with 3, 6 and 9 stories have been designed according to ECP-203 (2007) against gravity and seismic loads using ECP-201 (2012) (spectrum type 1 and 2). The analyses have been carried out using spectrum type 1 and 2 for each zone. The soil is considered soil class C and the reduction factor limited ductility of moment resisting frame, R, is taken equal 5. Software Sap2000 v18.2 [37] is utilized to create a 3-D finite element model, figure 5 for computation of the ultimate straining actions on slabs, beams and columns due to designed loads. The following points have been considered through the design process:

- The moment resisting frame type is considered sway type (for calculating effective length factor).
- The inter-story drift should not exceed 0.005 of the story height, h, as to verify the damage limitation requirements.
- The assumed steel ratio for the columns is varying from 1.0% to 1.4% relative to cross section area. In case the element capacity for axial load and biaxial bending does not satisfy the corresponding design value, the column section is increased keeping the same steel ratio.
- In case the ultimate resistance force provided by shear reinforcement does not satisfy the demand design value for the shear force, the specified stirrups for column are changed to satisfy such demand.
- The base code used for column design in software Sap2000 is BS8110. Modifications to some design parameters are implemented in order to be compatible with design requirement of the Egyptian code.
- The Design aids and examples, part 1 according to ECP-203 (2007) has been used to check the columns design.

For RC single-bay frame buildings with 3, 6 and 9 stories, tables 5, 6 and 7 summarize design column sections. In these tables, the design column sections are given for seismic zone intensity 0.25g using spectrum type 1 and 2. The cross section of beams is (25x50) for all buildings except the 9-stories building, seismic zone 0.25g spectrum type 2, where the cross section is (55x25) due to story-drift limitation. The steel reinforcements of beams are given in table 8 for each designed building.

For RC multi-bay frame buildings with 3, 6 and 9 stories, tables 9, 10 and 11 summarize design column sections. In these tables, the design column sections are given for two seismic zone intensity 0.15g and 0.25g using spectrum type 1 and 2. The cross section of beams is (25x50) for all buildings except the 9-stories building, seismic zone 0.25g spectrum type 2, where the cross section is (60x25) due to story-drift limitation. The steel reinforcements of beams are given in table 12 for each designed building.

The capacity/demand ratios for most columns are in lower stories of all the studied buildings and within the range from 0.75 to 0.90.

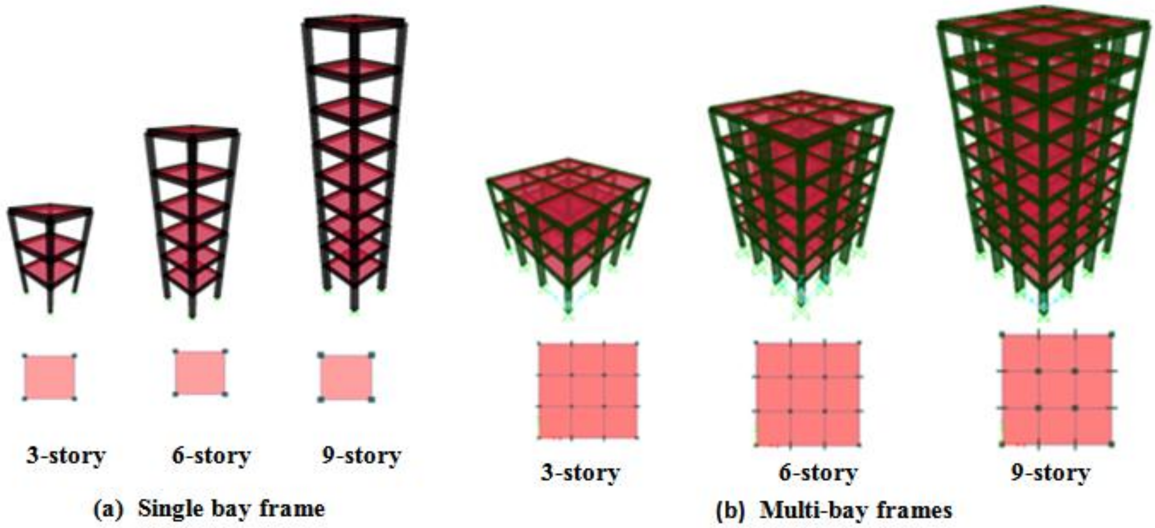


Fig. 5: 3D Finite Element Model

Table 5: Design Column sections for 9 story buildings, Single bay frame

Design zone	Spectrum type	Story number		
		(1), (2), (3)	(4), (5), (6)	(7), (8), (9)
		column	column	column
0.25g	1	40x40 (8 ϕ16)	35x35 (8 ϕ16)	30x30 (8 ϕ14)
	2	55x55 (12 ϕ18)	45x45 (12 ϕ16)	40x40 (8 ϕ16)

Table 6: Design Column sections for 6 story buildings, Single bay frame

Design zone	Spectrum type	Story number	
		(1), (2), (3)	(4), (5), (6)
		column	column
0.25g	1	35x35 (8 ϕ16)	30x30 (8 ϕ14)
	2	50x50 (12 ϕ16)	40x40 (8 ϕ16)

Table 7: Design Column sections for of 3 story buildings, Single bay frame

Design zone	Spectrum type	Story number
		(1), (2), (3)
		column
0.25g	1	30x30 (4 ϕ16+ 4ϕ14)
	2	35x35 (8 ϕ16)

Table 8: Reinforcement of Beams, Single bay frame

Design zone	Spectrum type	3 story building		6 story building		9 story building	
		Upper	Lower	Upper	Lower	Upper	Lower
0.25g	1	2 ϕ16+2 ϕ14	2 ϕ16+2 ϕ14	4 ϕ14	2 ϕ14+2 ϕ12	4 ϕ14	2 ϕ14+2 ϕ12
	2	2 ϕ16+2 ϕ14	2 ϕ16+2 ϕ14	4 ϕ16	2 ϕ16+2 ϕ14	5 ϕ16	4 ϕ16

Table 9: Design Column sections for 9 story buildings, Multi-bay frames

Design zone	Spectrum type	Story number					
		(1), (2), (3)		(4), (5), (6)		(7), (8), (9)	
		Interior column	Exterior column	Interior column	Exterior column	Interior column	Exterior column
0.25g	1	50x50 (12 ϕ16)	25x100 (16 ϕ16)	40x40 (8 ϕ16)	25x85 (14 ϕ16)	35x35 (8 ϕ16)	25x70 (12 ϕ16)
	2	65x65 (16 ϕ18)	25x125 (16 ϕ18)	50x50 (14 ϕ18)	25x100 (16 ϕ18)	40x40 (8 ϕ16)	25x80 (12 ϕ18)
0.15g	1	50x50 (12 ϕ16)	25x90 (12 ϕ16)	40x40 (8 ϕ16)	25x70 (10 ϕ16)	35x35 (8 ϕ16)	25x50 (8 ϕ16)
	2	55x55 (12 ϕ18)	25x100 (12 ϕ18)	45x45 (8 ϕ16)	25x85 (12 ϕ16)	35x35 (8 ϕ16)	25x70 (10 ϕ16)

Table 10: Design Column sections for 6 story buildings, Multi-bay frames

Design zone	Spectrum type	Story number			
		(1), (2), (3)		(4), (5), (6)	
		Interior column	Exterior column	Interior column	Exterior column
0.25g	1	45x45 (12 ϕ16)	25x70 (12 ϕ16)	40x40 (8 ϕ16)	25x60 (10 ϕ16)
	2	55x55 (12 ϕ18)	25x120 (22 ϕ18)	45x45 (12 ϕ16)	25x100 (12 ϕ16)
0.15g	1	45x45 (12 ϕ16)	25x65 (10 ϕ16)	35x35 (8 ϕ16)	25x50 (8 ϕ16)
	2	45x45 (8 ϕ18)	25x80 (14 ϕ16)	40x40 (8 ϕ16)	25x70 (10 ϕ16)

Table 11: Design Column sections for of 3 story buildings, Multi-bay frames

Design zone	Spectrum type	Story number (1), (2), (3)	
		Interior column	Exterior column
0.25g	1	35x35 (8 ϕ16)	25x60 (10 ϕ16)
	2	40x40 (8 ϕ16)	25x75 (14 ϕ16)
0.15g	1	35x35 (8 ϕ14)	25x45 (6 ϕ16)
	2	35x35 (8 ϕ16)	25x50 (8 ϕ16)

Table 12: Reinforcement of Beams, Multi-bay frames

Design zone	Spectrum type	3 story building		6 story building		9 story building	
		Upper	Lower	Upper	Lower	Upper	Lower
0.25g	1	4 ϕ16	3 ϕ16	4 ϕ16	3 ϕ16	5 ϕ16	4 ϕ16
	2	4 ϕ16	3 ϕ16	5 ϕ16	4 ϕ16	6 ϕ16	5 ϕ16
0.15g	1	3 ϕ16	2 ϕ16	3 ϕ16	2 ϕ16	2ϕ16+2 ϕ14	2ϕ16
	2	3 ϕ16	2 ϕ16	4 ϕ16	3 ϕ16	5 ϕ16	4 ϕ16

V. Cases Of Study

The following cases of study have been considered for RC frame buildings with 3, 6 and 9 stories:

1. Calculate base shear for Multi-bay frames (seismic zone intensity 0.15g and 0.25g) using ECP-201 (2012) (spectrum type 1 and 2) as well as IBC (2012).
2. Compare the empirical equation of fundamental period of vibration (T) given by the code and the accurate value calculated by SAP2000 program for Single and Multi-bay frames.
3. Perform nonlinear pushover static analysis to determine hinge status and corresponding base shear at yield and ultimate states for Single-bay frames (seismic zone intensity 0.25g) and Multi-bay frames (seismic zone intensity 0.15g and 0.25g) using ECP-201 (2012) (spectrum type 1 and 2).
4. Estimate response modification factor R for Single-bay frames (seismic zone intensity 0.25g) and Multi-bay frames (seismic zone intensity 0.15g and 0.25g) using ECP-201 (2012) (spectrum type 1 and 2).

VI. Results And Discussions

6.1 Base shear percent for Multi-bay frames using ECP-201 (2012) [spectrum type 1 and 2] as well as IBC (2012)

The base shear has been calculated for RC multi-bay frame buildings with 3, 6 and 9 stories. The soil is considered to be dense/stiff soil, which presents soil class C in ECP 2012 and soil class D in IBC 2012, table 13. The reduction factor, R, is taken equal 5. The analysis has been carried out for two seismic zone intensity 0.15g and 0.25g according to:

- ECP-201 (2012) [the spectrum is type 1]
- ECP-201 (2012) [the spectrum is type 2]
- IBC (2012).

For the above cases of analysis, base shear percent (Q design / own weight of building) have been plotted for RC multi-bay frame buildings with 3, 6 and 9 stories, figures 9 and 10. The results are plotted in figure 6 and summarized in table 14.

(i) With respect to the number of building stories (3, 6 and 9), base shear percent decreases with increasing the number of stories. This is valid for different seismic zone intensity as well as when using ECP-201 (2012) and IBC (2012).

(ii) With respect to spectrum type in ECP-201 (2012), base shear percent calculated for the studied building using spectrum type 1 are significantly smaller than those calculated spectrum type 2.

- For seismic zone intensity 0.25g, base shear percent for spectrum type 1 are 0.108, 0.068 and 0.051 for the three studied buildings (3, 6 and 9), while the corresponding percent for spectrum type 2 are 0.139, 0.123 and 0.093 respectively. The ratios of base shear percent for spectrum type 1 to type 2 are 0.78, 0.55, and 0.54 respectively.
- For seismic zone intensity 0.15g, base shear percent for spectrum type 1 are 0.065, 0.041 and 0.031 for the three studied buildings (3, 6 and 9), while the corresponding percent for spectrum type 2 are 0.085, 0.075 and 0.056 respectively. The ratios of base shear percent for spectrum type 1 to type 2 are 0.76, 0.55, and 0.55 respectively.

- The above results show that the decrease of the ratio of base shear percent for spectrum type 1 to type 2 are much pronounce for 6 and 9 story buildings. This is expected due to sharp down shape of spectrum type 1 started after time T_c (0.25-0.30 sec.).

(iii) With respect to ECP-201 (2012) spectrum type 2 and IBC (2012) spectrum, base shear percent calculated for the studied buildings using ECP-201 (2012) spectrum type 2 are slightly around those values calculated using IBC (2012).

- For seismic zone intensity 0.25g, base shear percent for ECP-201 (2012) spectrum type 2 are 0.108, 0.068 and 0.051 for the three studied buildings (3, 6 and 9), while the corresponding percent for spectrum IBC (2012) are 0.140, 0.111 and 0.083 respectively. The ratio of base shear percent for spectrum type 2 in ECP-201 (2012) and IBC (2012) are 0.99, 1.11, and 1.11 respectively.
- For seismic zone intensity 0.15g, base shear percent for ECP-201 (2012) spectrum type 2 are 0.065, 0.041 and 0.031 for the three studied buildings (3, 6 and 9), while the corresponding percent for spectrum IBC (2012) are 0.079, 0.076 and 0.056 respectively. The ratio of base shear percent for spectrum type 2 in ECP-201 (2012) and IBC (2012) are 1.07, 0.99, and 1.00 respectively.
- The above results show that the ratios of base shear percent for ECP-201 (2012) spectrum type 2 have small fluctuation compared to those values resultant from IBC (2012).

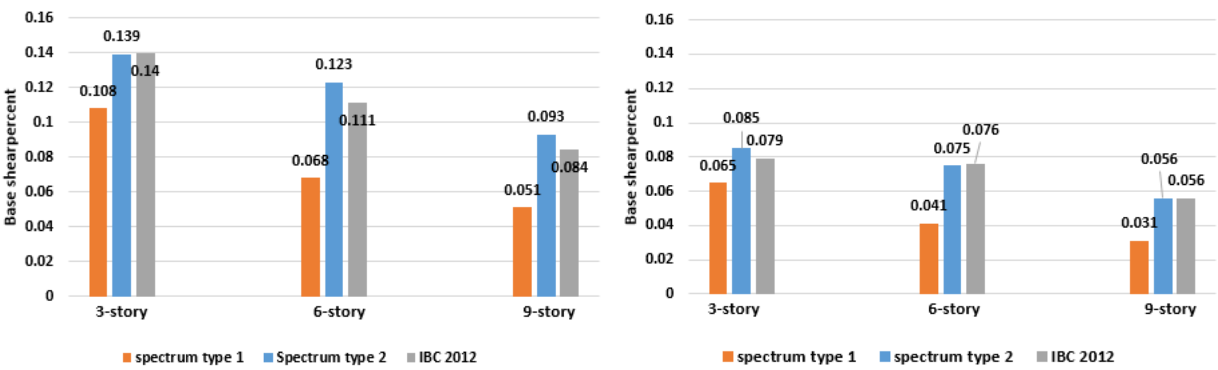
(v) The results in table 14 show the big difference in base shear percent according to the used type of spectrum. Thus, it is recommended to account for deep geology and revise the spectrum type especially for high seismic zones in Egypt [as recommended by EC8 (2004) for how to specify the type of spectrum].

Table 13: Ground Soil class C in ECP 2012 and similar Soil class in IBC 2012

Subsoil class	**Description of stratigraphic soil profile**	**Number of blows N_{SPT}**	**Undrained shear strength C_u (kN/m^2)**	**Shear wave velocity $V_{S,30}$ (m/sec)**
C (ECP 2012)	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters	15-50	250-70	360-180
D (IBC 2012)	Stiff soil with N_{SPT} or $\mathbf{C_u}$ or $\mathbf{V_{S,30}}$	15-50	100-50	360-180

Table 14: Base shear percent (Q_{design} / own weight of building) - ECP 2012 and IBC 2012

	ECP 2012		IBC 2012 ASCE7-10	ECP 2012		IBC 2012 ASCE7-10
Design zone	0.25g		0.25g	0.15g		0.15g
Spectrum type	1	2	-	1	2	-
3 story building	0.108	0.139	0.140	0.065	0.085	0.079
6 story building	0.068	0.123	0.111	0.041	0.075	0.076
9 story building	0.051	0.093	0.084	0.031	0.056	0.056



(a) Zone Intensity 0.25g **(b) Zone Intensity 0.15g**

Fig. 6: Base shear percent: ECP 2012 (spectrum type 1 and spectrum type 2) and IBC 2012, Multi-bay frames

Table 15 gives the ratio of the own weight of RC building (columns + beams +slab) designed using spectrum type 2 to the corresponding own weight using spectrum type 1. The results show that:

- For seismic zone intensity 0.15g, the ratio of the own weight of RC building (3, 6 and 9 stories) designed using spectrum type 2 to the corresponding own weight using spectrum type 1 are 1.01, 1.043 and 1.047 respectively.
- For seismic zone intensity 0.25g, the ratio of the own weight of RC building (3, 6 and 9 stories) designed using spectrum type 2 to the corresponding own weight using spectrum type 1 are 1.057, 1.109, and 1.180 respectively.
- The above results means that the maximum increase in the quantity of reinforcement concrete of the studied buildings depends on building height. In case of design using spectrum type 2 instead of spectrum type 1, the maximum increase for 9 story building reaches 4.7% and 18% in seismic zone intensity 0.15g and 0.25g respectively.

Table 15: Ratio of the own weight of RC building (columns + beams +slab) designed using spectrum type 2 to the corresponding own weight using spectrum type 1, Multi-bay frames

	Ratio for design zone 0.25g	Ratio for design zone 0.15g
3 story building	1.057	1.010
6 story building	1.109	1.043
9 story building	1.180	1.047

6.2 Fundamental natural period of the structures

Determination of the fundamental period of vibration (*T*) of a structure is essential in earthquake design. Standard design practices typically use code recommended empirical equations to estimate the design base shear. The current code equations (ECP (2012), EURO (2004)) provide the formulas or the approximate period of moment-resisting frames (MRFs), which are only dependent on the height of the structures.

$$T1 = C_t \ H^{3/4} \quad (6)$$

Where, C_t is 0.075 for moment resistance space concrete frames and H is the height of the building, in m.

The time period obtained from ECP-201 (2012) and SAP2000 (v18.2) is summarized in table 16 for one seismic zone intensity 0.25g for single bay frame and in table 17 for two seismic zone intensity 0.15g and 0.25g for multi bay frame using two types of spectrum (1 and 2). Ratios of calculated time period (program to code formula) presented in these tables have been drawn in figures 7 and 8.

From the above table and Figure, for all the building models, the fundamental period calculated from code formula is less than the one calculated by the analysis, the Eigen solution (SAP2000). This is due to the fact that in the analysis, partitions and cladding are not considered in the model and as a result the model is less stiff comparing with the real structure.

Table 16: Ratio of calculated time period (program to code formula), Single-bay frames

Design zone	0.25g					
Spectrum type	1			2		
Time period	Prog.	Code	ratio	Prog.	Code	ratio
3 story building	0.591	0.421	1.40	0.485	0.421	1.15
6 story building	0.916	0.682	1.34	0.754	0.682	1.10
9 story building	1.257	0.913	1.38	1.051	0.913	1.15

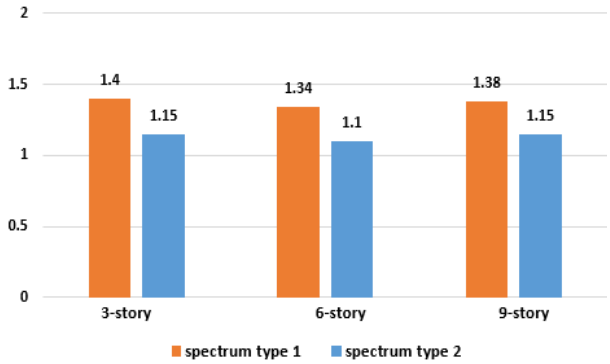
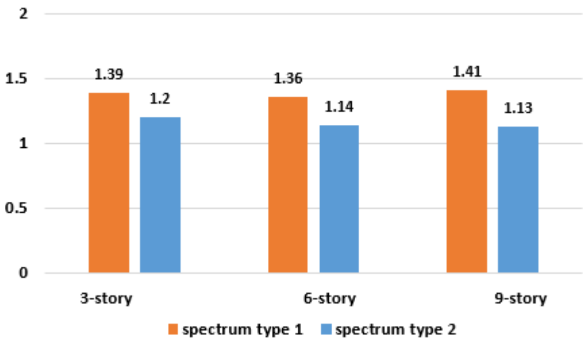


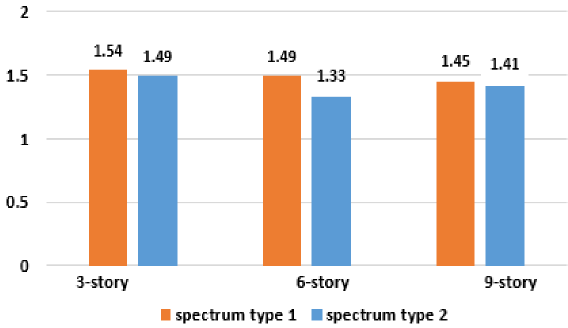
Fig. 7: Ratio of calculated time period (programme to code formula) zone=0.25g, Single-bay frames

Table 17: Ratio of calculated time period (programme to code formula), Multi-bay frames

Design zone	0.25g						0.15g					
Spectrum type	1			2			1			2		
Time period	Prog.	Code	ratio	Prog.	Code	ratio	Prog.	Code	ratio	Prog.	Code	ratio
3 story building	0.586	0.421	1.39	0.505	0.421	1.20	0.649	0.421	1.54	0.625	0.421	1.49
6 story building	0.930	0.682	1.36	0.778	0.682	1.14	1.020	0.682	1.49	0.906	0.682	1.33
9 story building	1.290	0.913	1.41	1.04	0.913	1.13	1.320	0.913	1.45	1.290	0.913	1.41



(a) Zone Intensity 0.25g



(b) Zone Intensity 0.15g

Fig. 8: Ratio of calculated time period (programme to code formula), Multi-bay frames

6.3 Base shear – roof displacement at yield and ultimate states of RC buildings using pushover analysis

Pushover analysis has been carried out for RC single and multi-bay frame buildings with 3, 6 and 9 stories using SAP2000 program in order to determine the performance level and deformation capacity (capacity curve) of the studied building. The formation of plastic hinges based on FEMA 356 rules are introduced as input into the SAP 2000 program. At every deformation step of the pushover analysis, the program determined the following.

(a) The position and plastic rotation of hinges in beams and columns.
(b) Hinges which have reached one of the three FEMA 356 rules IO, LS and CP limit states for hinge rotation, figure 9.
Columns isometric shape for hinge status at yield and ultimate states for all the studied buildings (3, 6 and 9 stories) have been determined.

Figures 10, 11 and 12 show the resulting pushover curves, in terms of base shear – roof displacement (V-Δ). In these figures, the slopes of the pushover curves are gradually reduced with increase of the lateral displacement of the building. This is due to the regressive formation of plastic hinges in beams and columns throughout the structure. The pushover curves reach a maximum point and afterwards there is a failure. The yield and ultimate shear bases and their corresponding roof displacement are determined and given below the plotted pushover curves for all the studied buildings in these figures.

For RC single and multi-bay frame buildings, the ratios of design base shear(EQ), Immediate Occupancy(IO) base shear and Life Safety (LS) base shear to Collapse Prevention (CP) base shear, using spectrum type 1 and 2 for seismic zone intensity 0.25g and 0.15g have been determined in tables 18 to 20. The results in these tables show that:

The ratios of Life Safety (LS) base shear to Collapse Prevention (CP) base shear of RC building (3, 6 and 9 stories) range between 0.98 to 0.99, for both seismic zone intensity 0.15g and 0.25g (spectrum type 1 and 2).
The ratios of Immediate Occupancy(IO) base shear to Collapse Prevention (CP) base shear of RC building (3, 6 and 9 stories) range between 0.88 to 0.94, for both seismic zone intensity 0.15g and 0.25g (spectrum type 1 and 2).
The ratios of design base shear(EQ) base shear to Collapse Prevention (CP) base shear of RC building (3, 6 and 9 stories) have different values depending on seismic zone intensity, spectrum type and number of story building. For seismic zone intensity 0.15g and 0.25g , spectrum type 2, the ratio values range between 0.42 to 0.68, while for spectrum type 1, the ratio values range between 0.34 to 0.52. These ratio values increase in almost cases, as the number of stories increases.

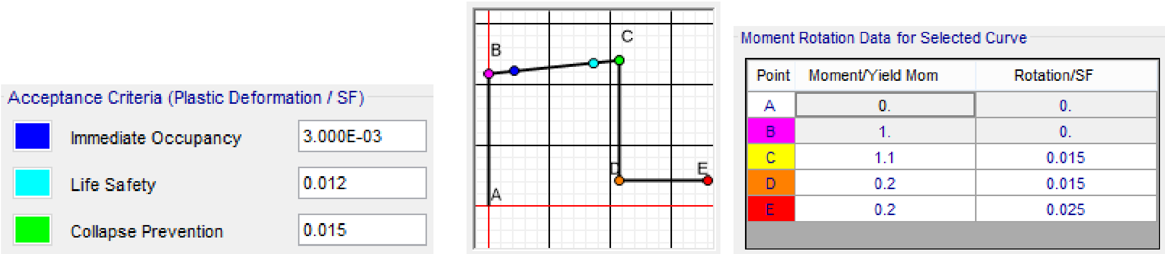


Fig. 9: The three rules Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) limit states for hinge rotation

Table 18: Ratios of design base shear (EQ), Immediate Occupancy (IO) base shear and Life Safety (LS) base shear to Collapse Prevention (CP) base shear, zone intensity, 0.25g, Single-bay frames

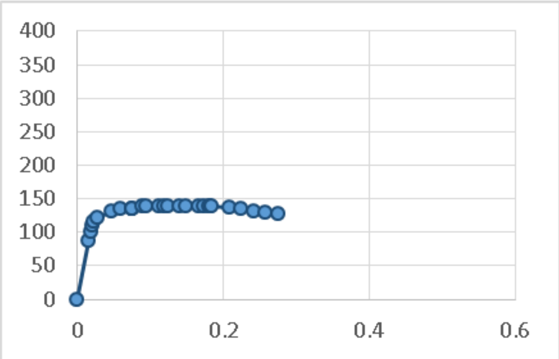
Ratio	[Design base shear(EQ) / Collapse Prevention (CP)]		Immediate Occupancy(IO) / Collapse Prevention (CP)]		[Life Safety (LS)/ Collapse Prevention (CP)]	
Spectrum type	1	2	1	2	1	2
3 story building	0.454	0.543	0.906	0.928	0.986	0.986
6 story building	0.478	0.583	0.940	0.930	0.987	0.983
9 story building	0.520	0.576	0.936	0.920	0.986	0.990

Table 19: Ratios of design base shear (EQ), Immediate Occupancy (IO) base shear and Life Safety (LS) base shear to Collapse Prevention (CP) base shear, zone intensity, 0.25g, Multi-bay frames

Ratio	[Design base shear(EQ) / Collapse Prevention (CP)]		Immediate Occupancy(IO) / Collapse Prevention (CP)]		[Life Safety (LS)/ Collapse Prevention (CP)]	
Spectrum type	1	2	1	2	1	2
3 story building	0.434	0.467	0.889	0.898	0.987	0.988
6 story building	0.466	0.544	0.930	0.910	0.982	0.992
9 story building	0.482	0.680	0.929	0.891	0.988	0.980

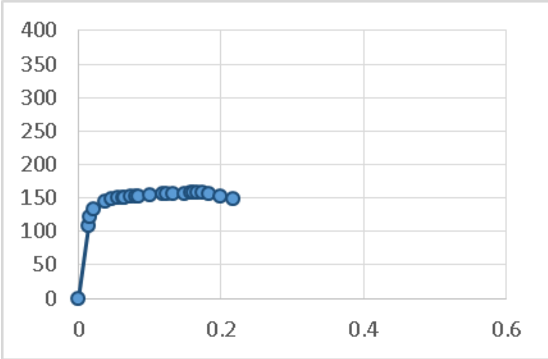
Table 20: Ratios of design base shear (EQ), Immediate Occupancy (IO) base shear and Life Safety (LS) base shear to Collapse Prevention (CP) base shear, zone intensity, 0.15g, Multi-bay frames

Ratio	[Design base shear(EQ) / Collapse Prevention (CP)]		Immediate Occupancy(IO) / Collapse Prevention (CP)]		[Life Safety (LS)/ Collapse Prevention (CP)]	
Spectrum type	1	2	1	2	1	2
3 story building	0.379	0.426	0.905	0.886	0.986	0.984
6 story building	0.372	0.486	0.909	0.908	0.989	0.990
9 story building	0.343	0.510	0.929	0.932	0.988	0.990



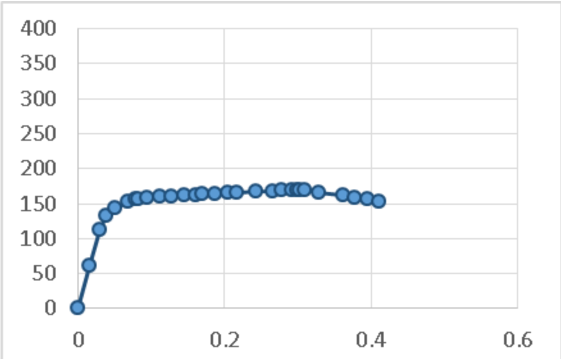
Du=0.072m Vu=136 kN Dy=0.0317m Vy=125 kN

(a) Spectrum type 1 , 3 story



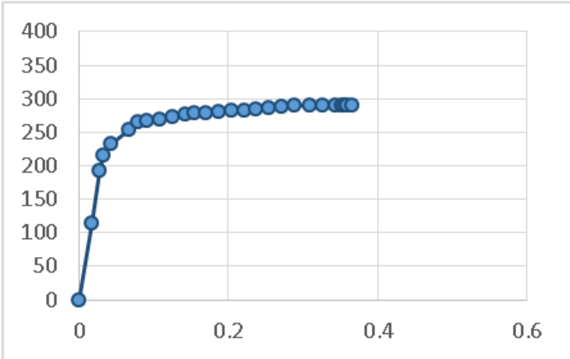
Du=0.1032m Vu=154 kN Dy=0.034m Vy=145 kN

(b)Spectrum type 2 , 3 story



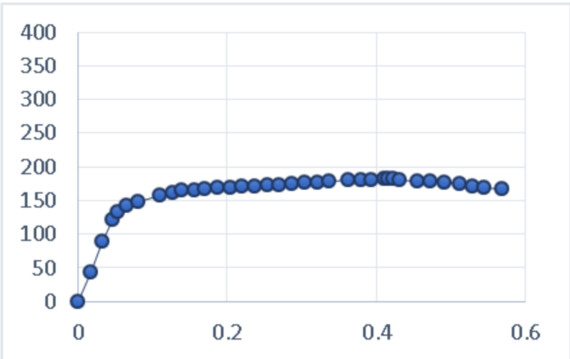
Du=0.217m Vu=165 kN Dy=0.105m Vy=160 kN

(c) Spectrum type 1 , 6 story



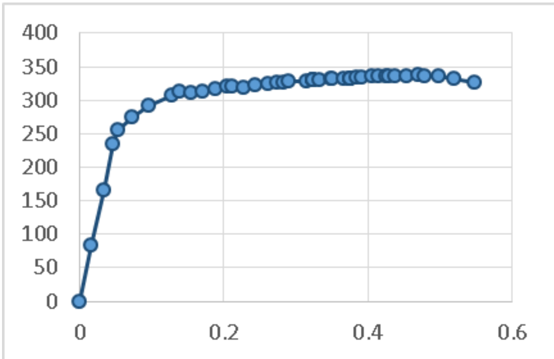
Du=0.236m Vu=283 kN Dy=0.0992m Vy=268 kN

(d) Spectrum type 2 , 6 story



Du=0.3117m Vu=176 kN Dy=0.157m Vy=167 kN

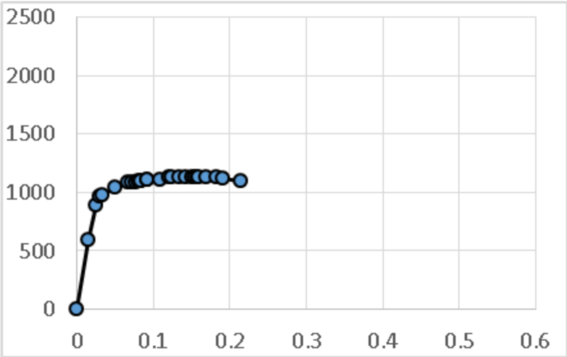
(e) Spectrum type 1 , 9 story



Du=0.349m Vu=331 kN Dy=0.1648m Vy=307 kN

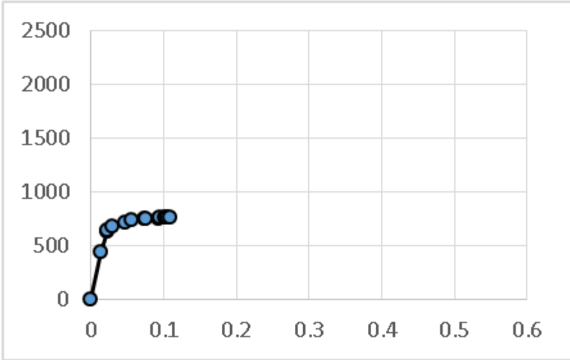
(f) Spectrum type 2 , 9 story

Fig.10: Pushover curves for the three buildings designed, zone 0.25g, Single-bay frames (x-axis, y-axis refer to: displacement (m), base shear (kN)).



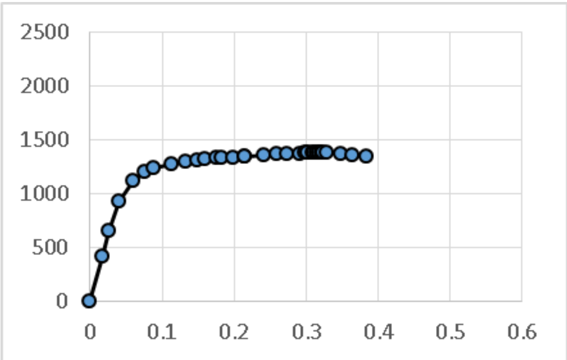
Du=0.0989m Vu=1108 kN Dy=0.0384m Vy=997 kN

(a) zone 0.25g , 3 story



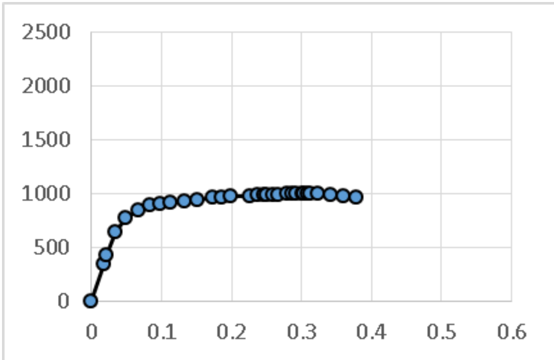
Du=0.0876m Vu=750.4 kN Dy=0.0345m Vy=689 kN

(b) zone 0.15g , 3 story



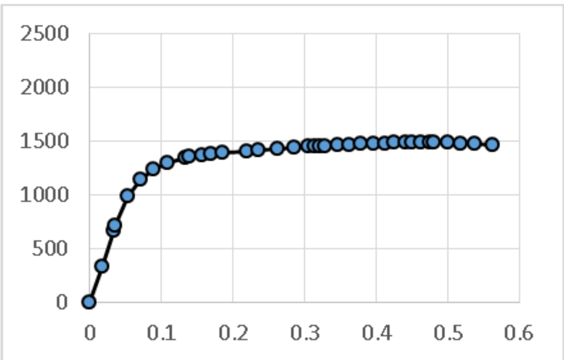
Du=0.2133m Vu=1346 kN Dy=0.0951m Vy=1249 kN

(c) zone 0.25g , 6 story



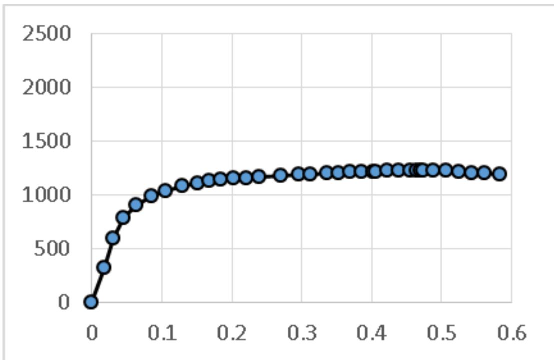
Du=0.2072m Vu=974 kN Dy=0.0903m Vy=895 kN

(d) zone 0.15g , 6 story



Du=0.3484m Vu=1462 kN Dy=0.1639m Vy=1376 kN

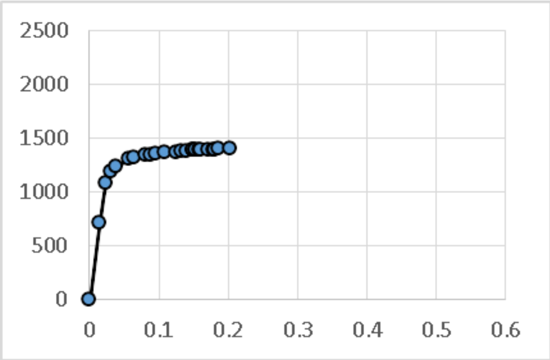
(e) zone 0.25g , 9 story



Du=0.3722m Vu=1212 kN Dy=0.1844m Vy=1141 kN

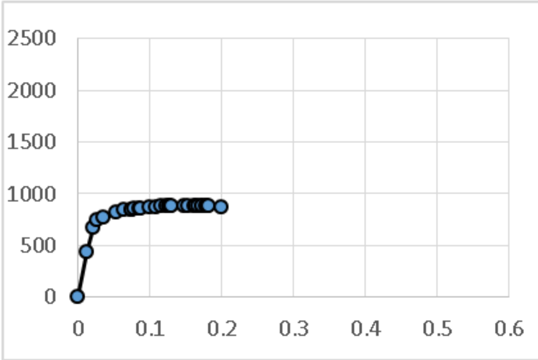
(f) zone 0.15g , 9 story

Fig. 11: Pushover curves for the three buildings designed using Spectrum type 1, Multi-bay frames (x-axis, y-axis refer to: displacement (m), base shear (kN)).



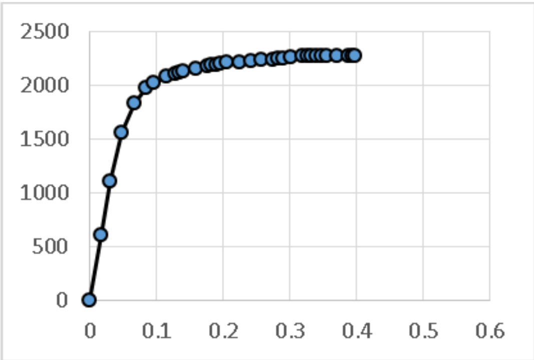
Du=0.1215m Vu=1370 kN Dy=0.0398m Vy=1245 kN

(a) zone 0.25g , 3 story



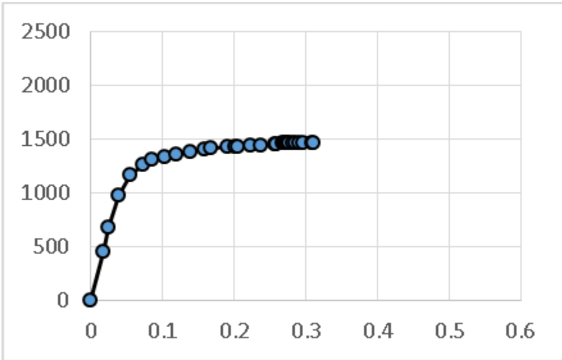
Du=0.099m Vu=866.5 kN Dy=0.038m Vy=780 kN

(b) zone 0.15g , 3 story



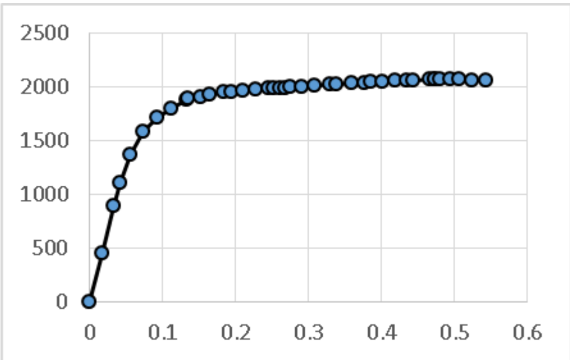
Du=0.2682m Vu=2243 kN Dy=0.1058m Vy=2058 kN

(c) zone 0.25g , 6 story



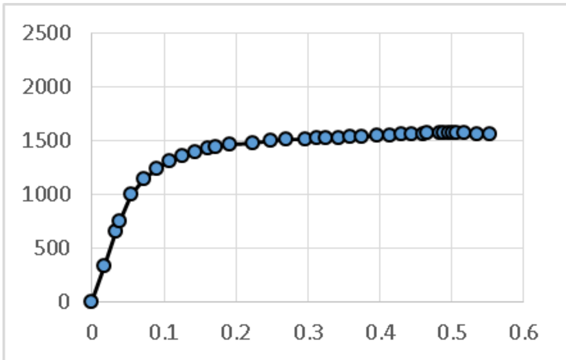
Du=0.2092m Vu=1433 kN Dy=0.0908m Vy=1314 kN

(d) zone 0.15g , 6 story



Du=0.340m Vu=2026 kN Dy=0.1211m Vy=1843 kN

(e) zone 0.25g , 9 story



Du=0.377m Vu=1545 kN Dy=0.186m Vy=1455 kN

(f) zone 0.15g , 9 story

Fig. 12: Pushover curves for the three buildings designed using Spectrum type 2, Multi-bay frames (x-axis, y-axis refer to: displacement (m), base shear (kN)).

6.4 Estimation of Response modification factor R

Equations 1 to 4 are used for estimating response modification factors from pushover curve results for all the studied buildings (3, 6 and 9 stories).

- For RC single-bay frame buildings, table 21 summarized the values of ductility ratio, over-strength factor and response modification factor in seismic zone intensity 0.25g. The response modification factor for spectrum type 1 and type 2 in seismic zone intensity 0.25g are plotted and compared in figure 13.
- For RC multi-bay frame buildings, tables 22 and 23 summarized the values of ductility ratio, over-strength factor and response modification factor in seismic zone intensity 0.25g and 0.15g respectively. The response modification factor for spectrum type 1 and type 2 in seismic zone intensity 0.25g and 0.15g are plotted and compared in figure 14.

The results in the above tables and figures show that the number of stories, seismic zone intensity and used spectrum type (1 or 2) significantly affect the response modification factor for the studied buildings (3, 6 and 9), single and multi- bay frames.

(i) For RC single-bay frame buildings,

- The values of response modification factor for seismic zone intensity 0.25g, spectrum type 2 are 4.28, 4.01 and 3.62 for the three studied buildings (3, 6 and 9), while the corresponding values for spectrum type 1 are 4.36, 4.26 and 3.80 respectively. This shows the significant effect of increasing the number of stories (building high) on decreasing the value of response modification factor.
- The values of response modification factor R in case of 3, 6 and 9 story buildings for seismic intensity 0.25g using spectrum type (1 or 2) are between 3.62 and 4.36. These values are less than the specified value of R as per ECP-201(2012) which equals 5.0 for limited ductility class for reinforced concrete moment frame structures.

(ii) For RC multi-bay frame buildings,

- The values of response modification factor for seismic zone intensity 0.25g, spectrum type 2 are 5.25, 4.64 and 4.04 for the three studied buildings (3, 6 and 9), while the corresponding values for spectrum type 1 are 5.26, 4.82 and 4.16 respectively. On the other hand, the values of response modification factor for seismic zone intensity 0.15g, spectrum type 2 are 5.62, 4.68 and 3.92 for the three studied buildings (3, 6 and 9), while the corresponding values for spectrum type 1 are 6.36, 6.09 and 5.78 respectively. This shows the significant effect of increasing the number of stories (building high) on decreasing the value of response modification factor.
- The values of response modification factor R in case of 6 and 9 story buildings for seismic intensities 0.25g and 0.15g using spectrum type 2 are between 3.92 and 4.68. These values are less than the specified value of R as in ECP-201(2012) which equals 5.0 for limited ductility class for reinforced concrete moment frame structures.

This means that the given value of R-factor at ECP-201(2012) is un-conservative value; as the accurate values of R-factor are less than the given value.

Table 21: Ductility ratio, μ, Over-strength factor, Ω and Response modification factor, R, seismic zone intensity 0.25g, Single bay frame

Spectrum type	1			2		
notation	**μ**	**Ω**	**R**	**μ**	**Ω**	**R**
3 story building	2.49	2.18	4.36	2.62	2.08	4.28
6 story building	2.07	2.06	4.26	2.37	1.69	4.01
9 story building	1.98	1.92	3.80	2.12	1.71	3.62

Table 22: Ductility ratio, μ, Over-strength factor, Ω and Response modification factor, R, seismic zone intensity 0.25g Multi-bay frames

Spectrum type	1			2		
notation	**μ**	**Ω**	**R**	**μ**	**Ω**	**R**
3 story building	2.57	2.27	5.26	3.05	2.11	5.25
6 story building	2.24	2.15	4.82	2.53	1.82	4.64
9 story building	2.03	2.04	4.16	2.80	1.44	4.04

Table 23: Ductility ratio, μ, Over-strength factor, Ω and Response modification factor, R, seismic zone intensity 0.15g, Multi-bay frames

Spectrum type	1			2		
notation	μ	**Ω**	**R**	μ	**Ω**	**R**
3 story building	2.56	2.60	6.36	2.61	2.30	5.62
6 story building	2.29	2.66	6.09	2.30	2.03	4.68
9 story building	2.02	2.86	5.78	2.02	1.93	3.92

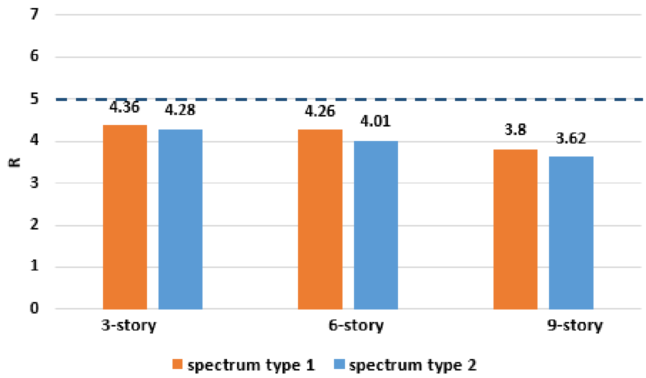
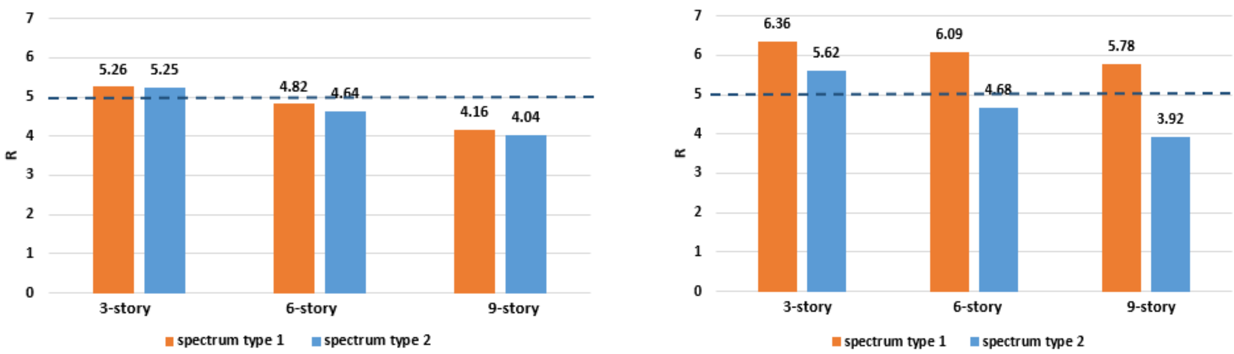


Fig. 13: Response Modification factors, zone=0.25g, Single-bay frames



(a) Zone Intensity 0.25g **(b) Zone Intensity 0.15g**

Fig. 14: Response Modification factors, Multi-bay frames

VII. Conclusions

- In this study, the response reduction factor (R) of RC limited ductility framed buildings is evaluated for both type of design response spectra specified in ECP-201 [2012]. Seismic and pushover analysis of RC frame buildings with 3, 6 and 9 stories designed according to ECP-203 (2007) have been performed using ECP-201 (2012) [spectrum type 1 and 2]. RC single-bay frame buildings in seismic zone 0.25g and multi-bay frame buildings in both seismic zone 0.15g and 0.25g has been studied. The significant outcomes of works are summarized as follows:

(I) Type of response spectra specified in ECP-201 [2012] and their corresponding design base shear for RC frame buildings

1. The design base shear according to ECP-201 (2012) spectrum type 2 is much bigger than those calculated using spectrum type 1. This is more pronouncing as the number of stories increases (building's height).
2. The design base shear according to ECP-201 (2012) spectrum type 2 is slightly around those calculated using IBC (2012).
3. It is recommended to account for deep geology and revise the spectrum type especially for high seismic zones in Egypt [EC8 (2004) state that spectrum type 1 is recommended in case expected surface-wave magnitude *Ms* > 5.5 and deep geology is not accounted for].
4. The ratio of the own weight of RC building (columns + beams +slab) designed using spectrum type 2 to the corresponding own weight using spectrum type 1 has been clarified for the studied buildings. It ranges from 1% to 18% depending on building height and seismic zone intensity. This in turn show the increase of cost if seismic design has been performed according to spectrum type 2 instead of spectrum type 1.

(II) Response reduction factor (R) of RC limited ductility framed buildings

5. The response reduction factor is considerably affected by the seismic zone and fundamental time period of the structure. It reduces as the seismic zone increases and increases as the fundamental time period increases.
6. The given value of R-factor at ECP-201(2012) equals 5.0 for limited ductility class of reinforced concrete moment frame structures is un-conservative value; as the accurate value of R-factor is less than the given value.
7. Recommended value of response reduction factor R for limited ductility class of limited ductility reinforced concrete moment frame structures in ECP-201(2012) is 3.9 for multi-story multi-bay frames and 3.6 for single bay multi-story frames.
8. It may be noted that Eurocode-8 (2004) specify values response reduction factor range between 3.0 and 3.9 for medium ductility reinforced concrete moment frames according to the frame configuration (One-story buildings, multi-story one-bay frames and multi-story multi-bay frames). UBC 97and IBC 2012 identify for RC ordinary frame buildings response reduction factor 3.5 and 3.0 respectively.

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