IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 17, Issue 1 Ser. V (Jan - Feb 2020), PP 01-14
www.iosrjournals.org

Seismic Response Modification factors for Multi-Story R.C buildings having Flat Slab System

Fayed. M. N[1], Nasr.E. Nasr[2,3], Saleh. M[4], Ahmed.G.M[5]
[1]*(Professor of Structural Engineering, Ain-Shams University, Egypt)*
[2]*(Associated Professor of Structural Engineering, Ain-Shams University, Egypt)*
[3]*(Vice Dean,Madina high institute for Engineering and technology, Egypt)*
[4]*(Assistant Professor of Structural Engineering, Ain-Shams University, Egypt)*
[5]*(Demonstrator of Structural Engineering, Egyptian Russian University, Egypt)*

ABSTRACT: *This paper is basically concerned with the calculation of the values of response modification factor at failure for idealized reinforced concrete multi-story flat slab system designed according to the Egyptian code of loads ECP-201 (2012). Parametric studies are carried out for RC multi-story flat slab with 3, 6 and 9 stories that are modelled in three-dimensions as residential buildings with various arrangements and variable parameters. SAP2000 software is used to be able to model and analyse these kinds of systems using three-dimensional nonlinear static pushover analysis considering of material and geometrical nonlinearity. The buildings are examined under the effect of several parameters such asnumber of stories, seismic zone intensity (0. 15g or 0. 25g) and type of spectrum (1 or 2) based on Egyptian code. Their influence on pushover curve, R-factor and its components are analysed. Comparisons among the results show the particular difference of some values and the indifference of other values including R-factor values.*

Date of Submission: 11-02-2020 Date of Acceptance: 26-02-2020

I. Introduction

Egypt is established in the north-eastern nook from the African Plate, alongside the south-eastern edge involving the Eastern Mediterranean place it is collaborating with all the Arabian and Eurasian Dishes through disparate and combined plate limits, respectively. Egypt is encompassed by about three active tectonic plate limitations: the African-Eurasian plate border, the Gulf of Suez-Red Sea plate boundary, alongside the Gulf of Aqaba-Dead Marine Transform Fault. A revised earthquake catalogue for Egypt is definitely due to the network and the relative movements between the plates regarding Eurasia, Africa and Persia. In the most recent 10 years, some areas in Egypt have been struck simply by significant earthquakes causing extensive damage. Such events have been interpreted as the consequence of this interaction, Abuo El-Ela et al, 2012.

According to ECP-201 [2012], Egypt is partitioned into five seismic zone which is based on expected surface-wave magnitude (Ms) at site, table 1. Typically the zoning guide appeared inside table 1 have probability of exceedance of 10 % in 50 years (return time of 475 years). The qualities obtained in the map should become scaled with the acceleration associated with gravity (9. 81 m/sec2). Two different spectra, centered on expected surface-wave degree (Ms) at site, such as in EC8 Type (I) and Type (II) usually are renamed in ECP-201 like type (2) and sort (1). ECP-201 specify with regard to coastal zones within the Mediterranean sea Sea (40 km range from shore), both reply spectrum curves, type (1) and type (2). For many other zones throughout Egypt (which include many regions with expected surface-wave magnitude Ms> 5.5), the response spectrum curve type (1) will be specified.

Table 1: Seismic zones of Egypt

zone	value of design ground acceleration (ag)
First Zone	0.10 g
Second Zone	0.125 g
Third Zone	0.15 g
Fourth Zone	0.20 g
Fifth Zone (a)	0.25 g
Fifth Zone (b)	0.30g

With respect to the capacity of structures in order to endure high levels associated with plastic deformations and pass energy, current building limitations design the structures to be able to withstand much lower makes than that are triggered by earthquakes to accomplish both safe and economical design. Current building rules uses single factor in order to reduce the forcesbrought on by earthquakes. This factor is named response modification factor (R-factor) in the Egyptian program code ECP-201 (2011), behavior aspect in Euro code EC8 (2003), and response adjustment coefficient in ASCE (2010). Building codes allocate the particular value of R-factor in order to structures according to several factors such as kind of the material employed in construction (i. e metallic, reinforced concrete, etc.), statical system, and ductility level. Anyway, this value serves a similar potential in every construction codes, this varies broadly from computer code to a different.

In this paper, RC limited ductility Flat slab buildings with 3, 6 and 9 stories have been designed according toECP-203 (2017)due to gravity and seismic loads using ECP-201 (2012) (spectrum type 1 and 2). The empirical equation of fundamental period of vibration (T) provided by the code and the accurate calculated by SAP2000 software 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 analysishas been performed to be able to determine hinge status and corresponding base shear from yield and ultimate areas. The response modification factor R for RC limited ductility flat slab buildings are evaluated in a couple of seismic zone intensity zero. 15g and 0. 25g using both type of design response spectra. The resultant values of response modification factor are compared with these values given in ECP-201 (2012). The results are summarized and discussed.

II. Concept For Determining Response Modification Factor

Typically, the factor represents to the nonlinear response of a composition by taking advantage involving the way that typically the structures possess significant hold strength and capacity to be able to dissipate energy, called above strength and ductility, correspondingly. Accordingly, the structure is definitely designed for a lot less bottom shear forces than might be required if typically the building is to stay elastic during serious trembling at a site. Some large reductions are typically due to two aspects: the ductility reduction component ($R\mu$)which minimizes the elastic demand push for the level of typically the most extreme yield quality including the structure, and typically the over-strength factor (Ω),which represents regularly the over-quality introduced in code-planned structures. Thus, the reply reduction factor (R) is usually:

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

The relation among the base-shear of the composition and its roof shift which can be considered by a nonlinear stationary analysis has been created in figure 1.

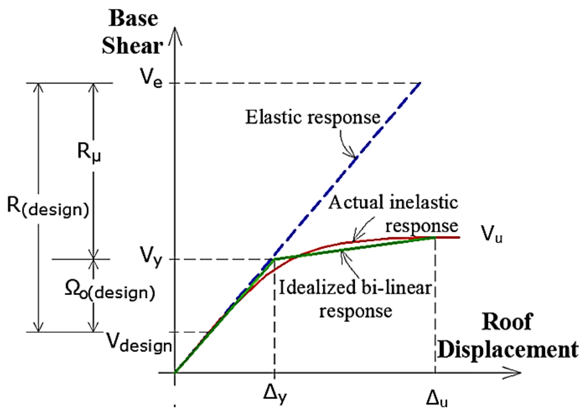


Figure 1: Relationship between power reduction factor (R), strength over-strength (Ω), in addition to ductility reduction factor ($R\mu$)

1. Over-strength factor Ω: The over-strength component (Ω) can end up being characterized as the percentage of the actual in order to design level strength (Elnashai and Mwafy, 2002). It might be communicated as:$\Omega = Vu / Vd$ (2)

Where Vu is the real strength and Vdis usually the design strength.

Typically the primary wellsprings in the strength over-strength results from effective yielding of critical parts, material over-strength, strain solidifying, capacity reduction factors, fellow member size, nonstructural elements in addition to special ductile detailing (Elnashai and Mwafy, (2002); Freeman, (1990); Lee et approach., (2005); Rodrigues et ing., (2013), Varum, (2003)).

2. Ductility reduction factor, Rμ: Normally the degree of inelastic twisting experienced by the quality framework exposed to the ground movement or the horizontal stackingis given by the displacement ductility ratio 'μ' (FEMA-451, (1999)). The inelastic behaviors of a structure may be idealized as:

$\mu = \Delta u / \Delta y$ (3)

Where μ is the displacement ductility ratio, Δu is the ultimate displacement and Δy is the yield displacement. Yielddisplacement and yield base shear are determinedthrough an idealization of the capacity curve.

Ductility reduction factor Rμis a function of structural characteristics such as ductility, lessening and fundamental period regarding vibration (T),and the particular characteristics of earthquake floor motion (Maheri and Akbari, (2003)). Researchers proposed different formulations in order in order to determine the ductility reduction factor Rμ, (Newmark and even Hall, (1973); Uang (1991), Paulay and Priestly, (1992), Miranda and Bertero, (1994); Kappos (1999), Priestley, (2000); Elnashai and Mwafy (2002), Mondal et al (2013)).

Throughout this study, the system proposed by Paulayin addition to Priestley (1992) is utilized.

$R\mu = 1.0$ 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)

WhereRμ is the ductility reduction factor and μ is the displacement ductility.

III. Nonlinear Static Pushover Analysis Method

1. Purpose ofpushover analysis

The purpose of pushover analysis is to be able to measure the expected functionality of a structural technique by evaluating its durability and deformation demands within designing earthquake resistant complexes by means of a new static inelastic analysis, in addition to comparing these demands in order to available capacities at the particular performance levels of fascination. The evaluation depends about an assessment of significant performance parameters, including international drift, inter-story drift, inelastic element deformations (either total or normalized with value to a yield value), deformations between elements, and even element and connection makes (for components and link associations that can't continue inelastic disfigurements). The inelastic fixed pushover analysis can get viewed as a way with regard to predicting seismic force plus deformation demands, which information in an estimated method for the redistribution associated with internal forces occurring if the structure is exposed to inertia forces of which no longer can get resisted within the supple range of structural habits. The pushover is anticipated to provide information in many response characteristics that will can't be gotten through an elastic static or even dynamic analysis, (Krawinklerainsi que al (1998)).

A pushover analysis is performed simply by exposing a structure to be able to a monotonically increasing routine of lateral loads, addressing the inertial forces which in turn would be through typically the structure when exposed to floor shaking. Under gradually improving loads various structural components may yield sequentially. Subsequently, at each event, typically the structure experiences a damage in stiffness. Using the pushover analysis, a typical nonlinear force displacement connection could be resolved.

2.Nonlinear Modelling of Building Elements

2.1 Nonlinear Modelling of RC Beam-Column Frame

The analytical model for a beam -column moment frame should represent strength, stiffness and deformation capacity of the beam -column joints along with potential failures due to flexure, shear and bond development. The connections between beams and columns should be represented by a stiff/rigid zone, having dimensions related to the geometric properties of beams and columns. The diaphragm action of the floor slab should be properly included in the model. Above-mentioned considerations are valid for nonlinear procedures as well as linear static, and dynamic procedures. For nonlinear procedures, beams and columns are recommended to be modelled using concentrated plastic hinge models or distributed plastic hinge models so that they are capable of representing inelastic response.

Performance-based engineering yields structures with predictable performance within defined levels of risk and reliability (FEMA 356 and ATC 40). 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. A representation of the monotonic load-deformation relationships are given in Figure 2.

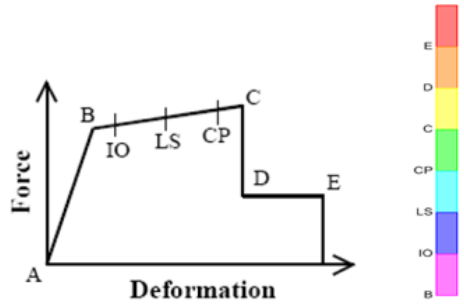


Figure 2: Typical load – deformation relation and target execution levels

The five points (A, B, C, D and E) are used to define the hinge rotation behaviour of RC members according to FEMA. Three additional points Immediate Occupancy (IO), Life Safety (LS) and (Collapse Prevention) CP, are used to define the acceptance criteria for the hinge. Numerous presentation goals for these levels, including the seismic change time frames, have been specified in Table 2. To analyze the cross-sections, Mander confined and unconfined concrete model (1988) and elasto-plastic steel model without hardening can used.

Table 2: 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	-

US code (ASCE 41-06, 2007) is utilizing a similar constraining qualities for the part plastic hinge turn interest for beams. However, the corresponding limiting values for columns have slightly changed.

3.2.2 Nonlinear Modelling of flat slab

Precise modelling for the nonlinear behaviour of reinforced concrete (RC) flat slab is an important task. Based on the principles of composite material mechanics, a multi-layer shell element model is proposed (Miao et al, 2006) to simulate the coupled in-plane/out-plane bending or the coupled in-plane bending-shear nonlinear behaviors of RC plate. The multi-layer shell element is based on the principles of composite material mechanics and it can simulate the coupled in-plane/out-plane bending and the coupled in-plane bending-shear nonlinear behaviors of RC flat slab. Basic principles of multi-layer shell element are illustrated by figure 3.

The multi-layer shell element is used for modelling of flat slab. The shell element is made up of many layers and material properties are assigned to various layers according to material constitutive law of concrete and steel.

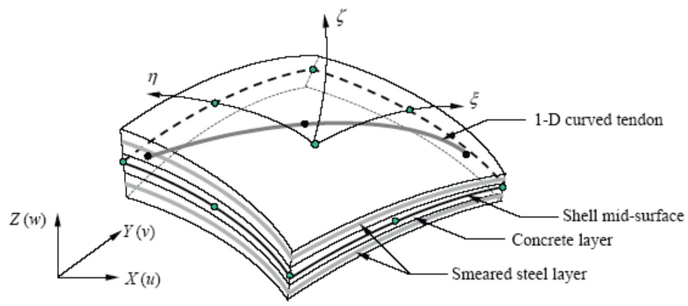
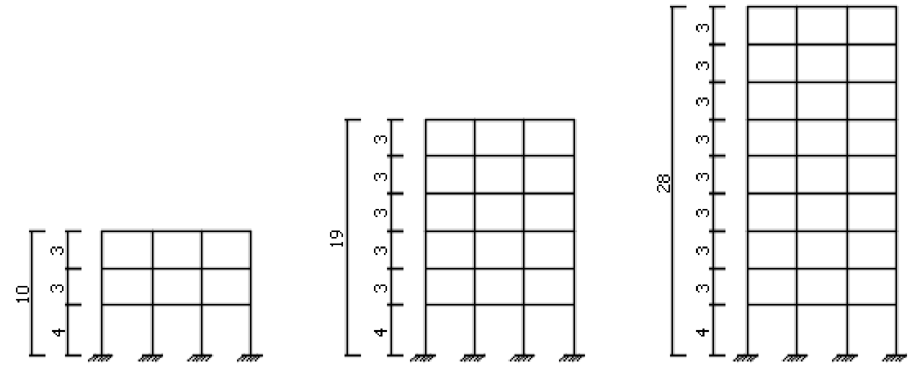


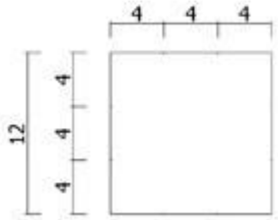
Figure 3: Multi-layer shell element.

IV. Characterization Of Building Models

Reinforced Concrete multi-story flat slab buildings with 3, 6 and 9 stories have been investigated utilizing SAP2000 (V20.1) auxiliary examination programming bundle (2016) The structures region unit displayed 3D flat slab structure using columns and shell element for flat slab (20 cm thickness) with inflexible floor diaphragms disseminate consistently the parallel loads on the vertical parts. Figure 4 shows elevation and plane layout for buildings dimensions.Material properties for reinforced Concrete buildings are represented in table 3. Stress-strain curves for concrete and, steel bars are illustrated in figure 5.



Elevation

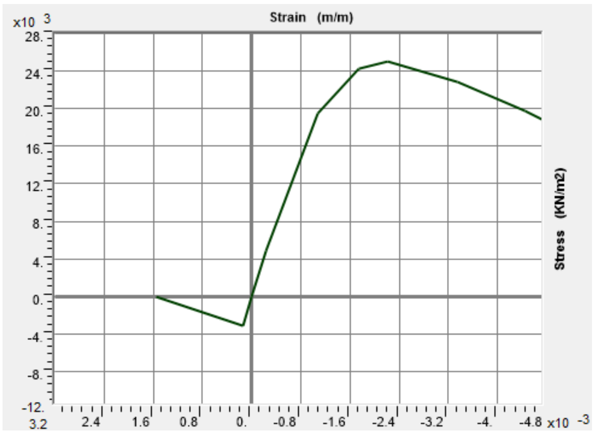


Plan

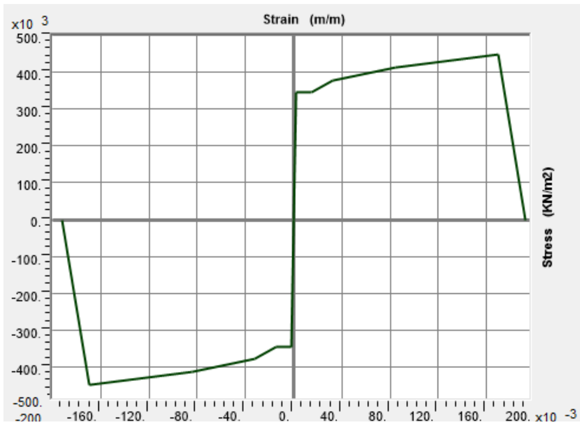
Figure 4: Layout of studied Flat slab buildings

Table 3: Material Properties for Buildings

Fc	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 bare

Figure5: Stress-strain curves introduced in SAP2000 (Computer & Structures Inc., 2011)

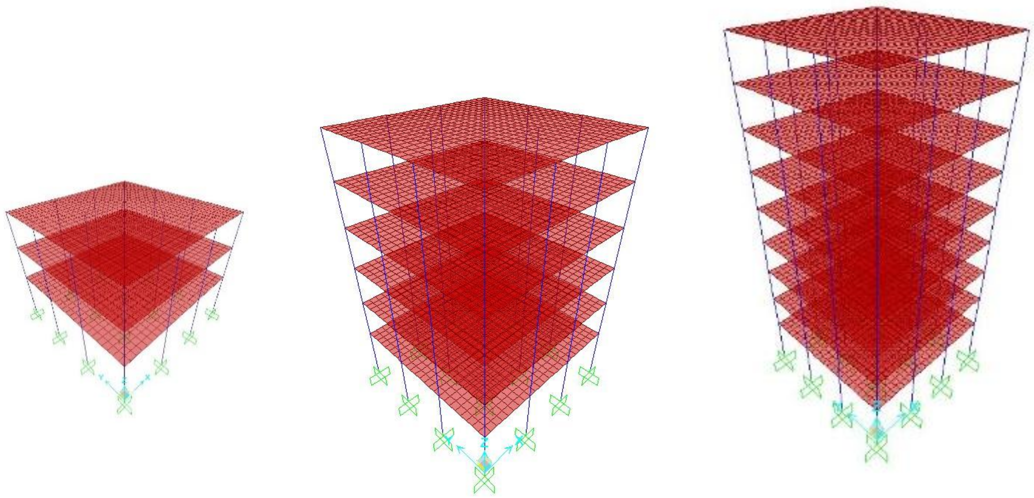
The followingloading cases have been considered:
1) Total Dead Load (D) is equal to DL+SDL+CL.
2) Dead Load (DL) is equal to the self-weight of the members and slabs.
3) Super-imposed Dead Load (SDL) equals to 3.0 kN/m². SDL includes partitions and ceiling weight.

4) Cladding Load (CL) is applied only on perimeter beams.
5) Live Load (L) equals to 2.0 kN/m².
The examined structures are exposed to various sorts of load combinations according to ECP 2012. These combinations are applied by the following terms:

$U = 1.40\ D + 1.60\ L$

$U = 1.12\ D + \alpha\ L \pm S$ (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 flat slab buildings with 3, 6 and 9 stories have been designed according to ECP-203 (2017) against gravity and seismic loads using ECP-201 (2012) and IBC (2012) for two seismic zone intensity 0.15g and 0.25g. The analysis 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 v20.1 is utilized to create a 3-D finite element model, figure 6 for computation of the ultimate straining actions on slab and columns due to designed loads. The following points have been considered through the design process.
The inter-story drift should not exceed 0.005 of the story height, h, as to verify the damage limitation requirements. The percent of the steel area divided to the column area is in range from 1.2% to 1.5% relative to cross section area.BS8110 is the codes which has been used in Sap 2000 with some modifications in design parameters to design the structure elements according to Egyptian code, ECP-203 (2017).



3 Stories – Flat Slab 6 Stories – Flat Slab 9 Stories – Flat Slab

Figure 6:3D Finite Element Flat slab buildings Models

For RC flat Slab buildings with 3, 6 and 9 stories, table 4,5 and 6 show design column sections by using ECP-201 (2012) . In these tables, the design column sections are given for seismic zone intensity 0.15g and 0.25g using spectrum type 1 and 2 for ECP-201 (2012) only. 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.

Table 4: Column sections for 3 story buildings

Design zone	Spectrum type	Story number	
		(1), (2), (3)	
		Interior column	Exterior column
0.25g	1	45x45 (12 ϕ18)	30x50 (12 ϕ16)
	2	40x40 (10 ϕ18)	30x80 (14 ϕ18)
0.15g	1	35x35 (8 ϕ18)	30x45 (8 ϕ18)
	2	35x35 (8 ϕ18)	30x50 (8 ϕ18)

Table 5: Column sections for 6 story buildings

Design zone	Spectrum type	Story number			
		(1), (2), (3)		(4), (5), (6)	
		Interior column	Exterior column	Interior column	Exterior column
0.25g	1	50x50 (14 ϕ18)	30x70 (14 ϕ18)	40x40 (8 ϕ18)	30x50 (8 ϕ18)
	2	60x60 (20 ϕ18)	40x150 (36ϕ18)	50x50 (12 ϕ18)	40x130 (20 ϕ18)
0.15g	1	45x45 (12 ϕ18)	30x50 (12 ϕ16)	35x35 (8 ϕ18)	30x30 (8 ϕ16)
	2	50x50 (14 ϕ18)	30x80 (18 ϕ16)	40x40 (8 ϕ16)	30x60 (10 ϕ18)

Table 6: Column sections for of 9 story buildings

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	60x60 (22 ϕ18)	30x70 (12 ϕ18)	50x50 (12 ϕ18)	30x50 (8 ϕ18)	40x40 (8 ϕ18)	30x30 (6 ϕ18)
	2	70x70 (28 ϕ18)	40x190 (48 ϕ18)	60x60 (16 ϕ18)	40x170 (38 ϕ18)	50x50 (8 ϕ16)	40x150 (24 ϕ18)
0.15g	1	50x50 (14 ϕ18)	30x60 (10 ϕ18)	40x40 (10 ϕ18)	30x40 (10 ϕ16)	35x35 (10 ϕ16)	30x30 (8 ϕ16)
	2	60x60 (20 ϕ18)	30x100 (18 ϕ18)	50x50 (12 ϕ18)	30x80 (10 ϕ18)	40x40 (12 ϕ16)	30x60 (8 ϕ16)

V. Cases Of Study

The following cases of study have been considered for RC flat slab structures with 3, 6 and 9 stories:

1. Calculate base shear (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 the studied structures.
3. Perform nonlinear pushover static analysis to determine hinge status and corresponding base shear at yield and ultimate states (seismic zone intensity 0.15g and 0.25g) using ECP-201 (2012) (spectrum type 1 and 2).
4. Calculate the response modification factor R for flat slab structures with 3, 6 and 9 stories (seismic zone intensity 0.15g and 0.25g) using ECP-201 (2012) (spectrum type 1 and 2).

VI. Results And Discussions

1.Base shear percent for Multi-bay flat slab buildings using ECP-201 (2012) [spectrum type I and II] as well as IBC (2012)

The base shear has been calculated for R.C. flat slab buildings with 3, 6 and 9 stories. The soil is viewed as dense/stiff soil, which presents soil class C in ECP 2012 and soil class D in IBC 2012, table 7. The reduction factor, R, is taken equal 5. The analysis has been carried out for two seismic zone intensity 0.15g and 0.25g as per:

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

For the above cases of analysis, base shear percent (Q design / own weight of building) have been plotted for flat slab buildings with 3, 6 and 9 stories, the outcomes are plotted in figure 7 and condensed in table 8.

(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 using ECP-201 (2012), base shear percent for spectrum type 1 are 0.216, 0.134 and 0.093 for the three studied buildings (3, 6 and 9), while the corresponding percent for spectrum type 2 are 0.22, 0.202 and 0.152 respectively.
- For seismic zone intensity 0.15g using ECP-201 (2012), base shear percent for spectrum type 1 are 0.133, 0.083 and 0.057 for the three studied buildings (3, 6 and 9), while the corresponding percent for spectrum type 2 are 0.138, 0.134 and 0.093 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 for flat slab buildings. This is expected due to sharp down shape of spectrum type 1 started after time Tc (0.25-0.30 sec.).

(iii) The results in table 8 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].

(v) With respect to IBC (2012) spectrum:

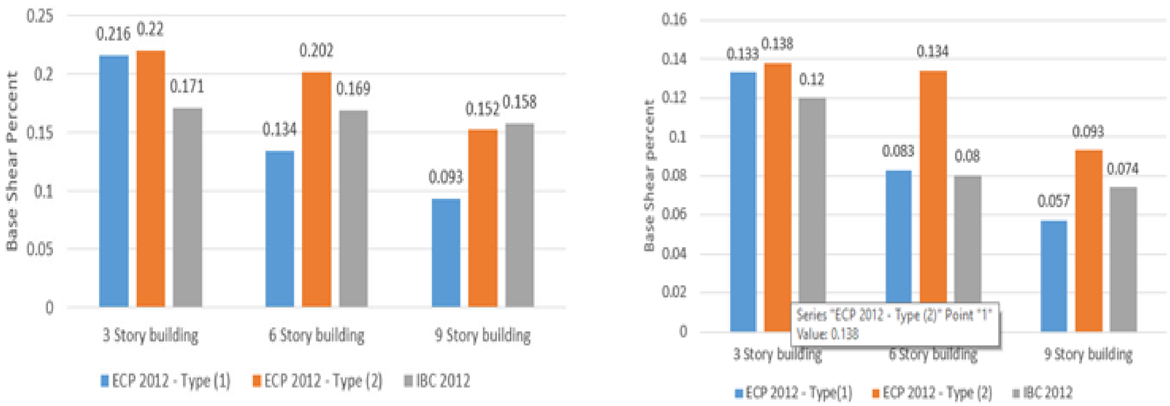
- For seismic zone intensity 0.25g using IBC (2012), base shear percent are 0.171, 0.169 and 0.158 for the three studied buildings (3, 6 and 9).
- For seismic zone intensity 0.15g using IBC (2012), base shear percent are 0.120, 0.080 and 0.113 for the three studied buildings (3, 6 and 9).

Table 7: Ground Soil class C in ECP 2012 and similar Soil class in IBC 2012

Subsoil class	Description of stratigraphic soil profile	Number of blows NSPT	Undrained shear strength Cu (kN/m2)	Shear wave velocity VS,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 metres	15-50	250-70	360-180
D (IBC 2012)	Stiff soil with NSPT or Cu or VS,30	15-50	100-50	360-180

Table 8: Base shear percent (Qdesign / own weight of building) - ECP 2012 and IBC 2012

	ECP 2012				IBC 2012 ASCE7-10	
Design Zone	0.25g		0.15g		0.25g	0.15g
Spectrum type	1	2	1	2	-	-
3 Story building	0.216	0.220	0.133	0.138	0.171	0.120
6 Story building	0.134	0.202	0.083	0.134	0.169	0.080
9 Story building	0.093	0.152	0.057	0.093	0.158	0.074



(a) Zone Intensity 0.25g – Flat slab buildings (b) Zone Intensity 0.15g – Flat slab buildings

Figure 7: Base shear percent: ECP 2012 (spectrum type 1 and spectrum type 2) and IBC 2012

Table 9 gives the ratio of the own weight of flat slab building designed using spectrum type 2 to the corresponding own weight using spectrum type 1 for ECP 2012.The results show that:

- For seismic zone intensity 0.25g, the ratio of the own weight of flat slab buildings (3, 6 and 9 stories) designed using spectrum type 2 to the corresponding own weight using spectrum type 1 are 1.301, 1.304 and 1.394 respectively.
- For seismic zone intensity 0.15g, the ratio of the own weight of flat slab buildings (3, 6 and 9 stories) designed using spectrum type 2 to the corresponding own weight using spectrum type 1 are 1.040, 1.089 and 1.125 respectively.
- The above results means that the maximum increase in the quantity of reinforcement concrete of the flat slab 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 26.1% and 7.56% in seismic zone intensity 0.25g and 0.15g respectively.
-

Table 9: 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

	Flat Slab buildings	
	Ratio for design zone 0.25g	Ratio for design zone 0.15g
3 story building	1.301	1.040
6 story building	1.304	1.089
9 story building	1.394	1.125

2. Fundamental natural period of the structures

Determination of the fundamental period of vibration (T) of a structure is essential in earthquake design. Standard structure rehearses ordinarily use code prescribed experimental conditions to evaluate the design base shear. The current code equations (ECP (2012), EURO (2004)) and (IBC 2012) provide the formulas or the approximate period of moment-resisting frames (MRFs), which are only subject to the stature of the structures.

T1 = Ct H3/4 (6)

Where, Ct is 0.075 for moment resistance space concrete frames and H is the height of the building, in m. Ct is 0.05 for flat slab buildings.

The time period obtained from ECP-201 (2012) and SAP2000 (v20.1) is outlined in table 10for two seismic zone intensity 0.15g and 0.25g for flat slab buildingsusing two types of spectrum (1 and 2). Ratios of calculated time period (program to code formula) presented in these tableshave been drawn in figures8 and 9.

From the table10, figures 8 and 9, for all the building models, the time periods obtained from the code have higher values than the Eigen solution (SAP2000). This demonstrates code-based techniques overestimate the principal time of vibration of structures. The fundamental period calculated from code formula is less than the one calculated by the analysis. 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 10: Ratio of calculated time period (program to code formula).

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	0.682	0.281	2.43	0.631	0.281	2.25	0.769	0.281	2.63	0.769	0.281	2.74
6 story building	1.265	0.455	2.78	0.966	0.455	2.12	1.381	0.455	3.04	1.237	0.455	2.72
9 story building	1.860	0.609	3.05	1.501	0.609	2.46	2.031	0.609	3.33	1.828	0.609	3

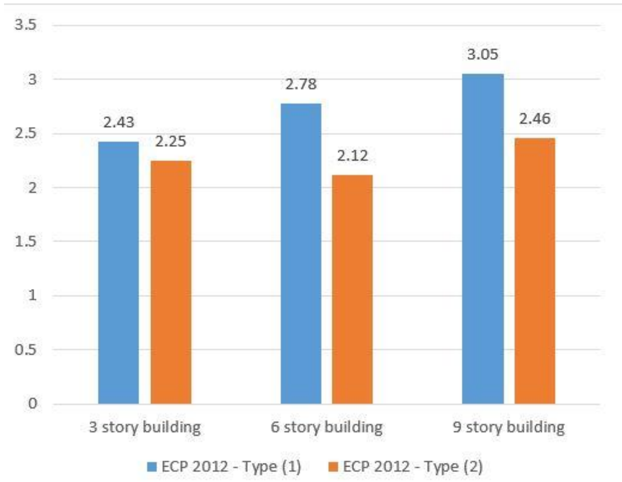


Figure 8: Ratio of calculated time period (programme to code formula) zone=0.25g.

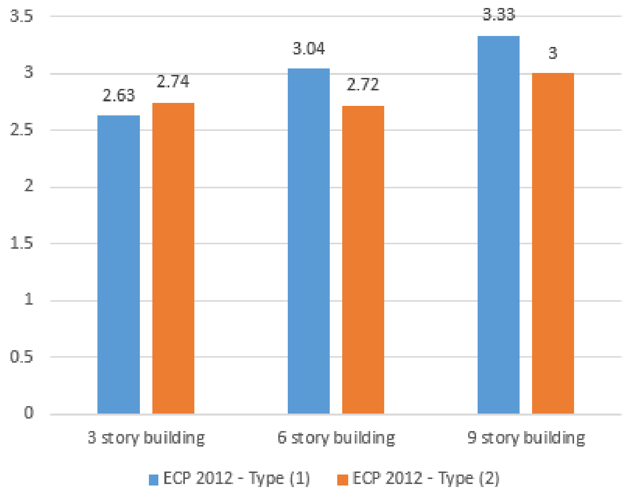


Figure 9: Ratio of calculated time period (programme to code formula) zone=0.15g.

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

Pushover analysis has been carried out for flat slab 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 development of plastic hinges dependent on FEMA 356 guidelines are brought as contribution to 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 foundation and columns.

(b) Hinges which have arrived at one of the three FEMA 356rules IO, LS and CP limit states for hinge rotation, figure 10.

For RC flat slab 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 11and12. The results in these tables show that:

- The ratios of Life Safety (LS) base shear to Collapse Prevention (CP) base shear of RC
Flat slab building (3, 6 and 9 stories) range between 0.622 to 0.947, 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 flat slab building (3, 6 and 9 stories) range between 0.422 to 0.588, 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 flat slab ratio, the ratio values range between 0.269 to 0.573.

These ratio values increase in almost cases, as the number of stories increases.

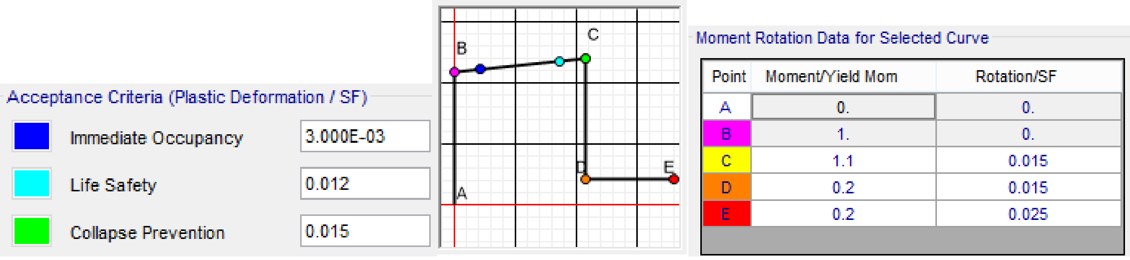


Figure 10: The three rules Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) limit states for hinge rotation

Table 11: 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.

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	0.373	0.363	0.493	0.442	0.872	0.865
6 story building	0.408	0.499	0.535	0.525	0.881	0.877
9 story building	0.407	0.527	0.588	0.569	0.890	0.889

Table 12: 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.

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	0.269	0.326	0.571	0.494	0.879	0.869
6 story building	0.275	0.397	0.526	0.483	0.886	0.879
9 story building	0.283	0.456	0.545	0.500	0.893	0.892

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 multi-bay flat slab buildings using ECP 201 (2012), tables from 16 to21summarized 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 figure11.

The results in thebelowtables 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), multi-bay flat slab.

- The values of response modification factor for seismic zone intensity 0.25g, spectrum type 2 are 5.67, 4.81 and 3.63 for the three studied buildings (3, 6 and 9), while the corresponding values for spectrum type 1 are 5.83, 5.07 and 4.80 respectively. On the other hand, the values of response modification factor for seismic zone intensity 0.15g, spectrum type 2 are 6.20, 5.78 and 4.54 for the three studied buildings (3, 6 and 9), while the corresponding values for spectrum type 1 are 6.85, 6.39 and 5.41 respectively. This shows the significant effect of increasing the number of stories (building high) on decreasing the value of response modification factor.

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. 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 13: Ductility ratio, μ by using $\mu = \Delta u / \Delta y$, seismic zone intensity 0.25g

Spectrum type	1			2		
notation	Δu	Δy	μ	Δu	Δy	μ
3 story building	0.122	0.056	2.177	0.105	0.051	2.055
6 story building	0.31	0.15	2.069	0.361	0.151	2.399
9 story building	0.528	0.27	1.952	0.539	0.279	1.931

Table 14: Over-strength factor, Ω by using $\Omega = Vu / Vd$,seismic zone intensity 0.25g

Spectrum type	1			2		
notation	Vu	Vd	Ω	Vu	Vd	Ω
3 story building	1716	640.8	2.678	1859	674	2.759
6 story building	1974	806.2	2.449	3174	1584	2.004
9 story building	2074	844.4	2.456	3640	1935	1.881

Table 15: Ductility ratio, μ, Over-strength factor, Ω and Response modification factor, R, seismic zone intensity 0.25g

Spectrum type	1			2		
notation	μ	Ω	R	μ	Ω	R
3 story building	2.177	2.678	5.83	2.055	2.759	5.67
6 story building	2.069	2.449	5.07	2.399	2.004	4.81
9 story building	1.952	2.456	4.80	1.931	1.881	3.63

Table 16: Ductility ratio, μ by using μ = Δu / Δy, R, seismic zone intensity 0.15g

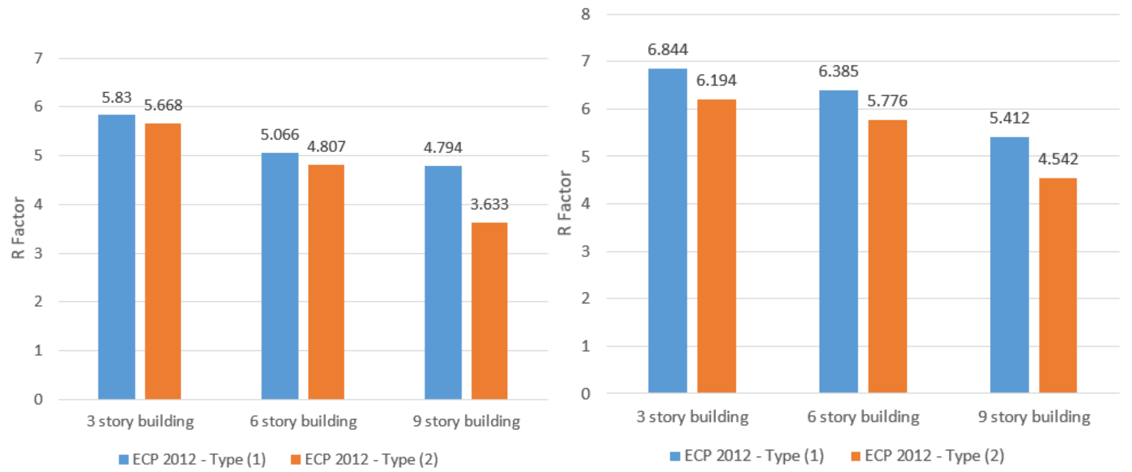
Spectrum type	1			2		
notation	Δu	Δy	μ	Δu	Δy	μ
3 story building	0.095	0.042	2.233	0.115	0.064	1.788
6 story building	0.305	0.174	1.758	0.327	0.142	2.296
9 story building	0.339	0.221	1.537	0.504	0.243	2.073

Table 17: Over-strength factor, Ω by using Ω = Vu / Vd, seismic zone intensity 0.15g

Spectrum type	1			2		
notation	Vu	Vd	Ω	Vu	Vd	Ω
3 story building	1186	387	3.065	1295	374	3.463
6 story building	1692	466	3.632	2060	819	2.516
9 story building	1716	487	3.522	2114	965	2.191

Table 18: Ductility ratio, μ, Over-strength factor, Ω and Response modification factor, R, seismic zone intensity 0.15g

Spectrum type	1			2		
notation	μ	Ω	R	μ	Ω	R
3 story building	2.233	3.065	6.85	1.788	3.463	6.20
6 story building	1.758	3.632	6.39	2.296	2.516	5.78
9 story building	1.537	3.522	5.41	2.073	2.191	4.54



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

Figure 11: Response Modification factors

VII. Conclusions

In this examination, the response reduction factor (R) of RC flat slab buildings isassessed for both sort of design response spectra specified in ECP-201 [2012]. Seismic and pushover analysis of flat slab buildings with 3, 6 and 9 stories designed according to ECP-203 (2017)have been performed utilizing ECP-201 (2012) [spectrum type 1 and 2].The critical results of works are condensed as pursues:

1. The design base shear according to ECP-201 (2012) spectrum type 2is 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 2is 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 1has been clarified for the studied buildings. It ranges from 3% to 15% 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.

5. Typically the response reduction factor is usually considerably affected by the particular seismic zone and basic period of time of the composition. It reduces as typically the seismic zone increases plus increases as the important time period increases.
6. The given value of R-factor at ECP-201(2012) equals 5.0 is un-conservative value for flat slab buildings; as the accurate value of R-factor is less than the given value.

References

[1]. Abuo El-Ela A., El-Hadidy, M. and AbouElenean, K. (2012) "Seismic hazard studies in Egypt" NRIAG Journal of Astronomy and Geophysics Volume 1, Issue 2.

[2]. ASCE SEI/ASCE 7–10 (2010), Minimum Design Loads for Buildings and Other Structures, Reston (USA): American Society of Civil Engineers.

[3]. Asgarian, B. and Shokrgozar, H.R. (2009) BRBF response modification factor, Journal of Constructional Steel Research 65, 290_298.

[4]. ATC 19 (1995), Seismic Response Modification Factors, Applied Technical Council, California Seismic Safety Commission, Redwood City, California.

[5]. ATC 40 (1996), Seismic Evaluation and Retrofit of Concrete Buildings, Applied Technical Council, California Seismic Safety Commission, Redwood City, California.

[6]. Borzi B and Elnashai AS (2000), "Refined Force Reduction Factors for Seismic Design," Engineering Structures, 22(10): 1244−1260.

[7]. Chaulagain, H., Rodrigues, H., Spacone, E., Guragain, R., Mallik, R. and Varum, H. (2014) "Response reduction factor of irregular RC buildings in Kathmandu valley" EarthqEng&EngVib (2014) 13: 455-470.

[8]. EC8 (2004), "Design of structures for earthquake resistance" Part 1: General rules, Seismic action and rules for buildings. European Committee for Standardization, Brussels, Belgium.

[9]. ECP (2012) ECP-201: Egyptian code for calculating loads and forces in structural work and masonry. Housing and Building National Research Center. Ministry of Housing, Utilities and Urban Planning, Cairo.

[10]. ECP (2017) ECP-203: Egyptian code of practice for design and construction of concrete structures, Housing and Building National Research Center, Ministry of Housing, Utilities and Urban Planning, Cairo.

[11]. Elnashai, A.S. and Mwafy, A.M. (2002) "Overstrength and force reduction factor of multi-story reinforced concrete buildings" The structural design of Tall Buildings, 11(5), 329-351.

[12]. Elnashai,A.S and A.M. Mwafy, (2002) "Calibration of Force reduction factors for RC Building" Journal of Earthquake Engineering, Vol. 6, No. 2, 239-273.

[13]. FEMA 356 (1997), Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, DC.

[14]. IBC (2012) International building Code, International Code Council.

[15]. Jain, S. K. and Navin, N., (1995), "Seismic Overstrength in Reinforced Concrete Frames", Journal of Structure Engineering, ASCE, Vol. 121, No. 3, pp. 580-585.

[16]. Krawinkler, H. and Seneviratna, G. D. (1998) "Pros & Cons of Pushover Analysis", Engineering Structures, Vol. 20, No. 4-6, pp. 452-464.

[17]. Lee DG, Cho SH and Ko H (2005), "Response Modification Factors for Seismic Design of Building Structures in Low Sesimicity Regions," Korea Earthquake Engineering Research Center.

[18]. Maheri, M.R. and Akbari, R. (2003). "Seismic behaviour factor, R, for steel X– braced and knee-braced RC buildings", Engineering Structures, 25, 1505-1513.

[19]. Maheri, M.R. and Akbari, R. (2003). "Seismic behaviour factor, R, for steel X– braced and knee-braced RC buildings", Engineering Structures, 25, 1505-1513.

[20]. Mander, J. B., Priestley, M. J. N. and Park, R. (1988a), "Observed stress-strain behavior of confined concrete," J. Struct. Engrg., ASCE, Vol. 114 No. 8, pp. 1827-1849.

[21]. Mander, J. B., Priestley, M. J. N. and Park, R.(1988b) "Theoretical stress-strain model for confined concrete," J. Struct. Engrg., ASCE, Vol. 114, No. 8, pp. 1804-1826.

[22]. Miao, Z. W. Lu1, X. Z. Jiang1, J. J. and Ye, L. P. (2006) " Nonlinear FE Model for RC Shear Walls Based on Multi-layer Shell Element and Microplane Constitutive Model Computational methods in Engineering and Science EPMESC, Sanya, Hainan,China.

[23]. Miranda E and Bertero VV (1994), “Evaluation of Strength Reduction Factors for Earthquake-resistant Design,” Earthquake Spectra, 10(2): 357−379.
[24]. Mondal A, Ghosa S and Reddy G (2013), “Performance based Evaluation of the Response Reduction Factor for Ductile RC Frames,” Engineering structures, 56: 1808
[25]. Newmark NM and Hall WJ (1973), “Seismic Design Criteria For Nuclear Reactor Facilities,” Report No 46, Building Practices for Disaster Mitigation, National Bureau of Standards, U.S. Department of Commerce.
[26]. Newmark NM and Hall WJ (1973), “Seismic Design Criteria For Nuclear Reactor Facilities,” Report No 46, Building Practices for Disaster Mitigation, National Bureau of Standards, U.S. Department of Commerce.
[27]. Paulay T and Priestley MJN (1992), Seismic Design of Reinforced Concrete and Masonry Buildings, Wiley Interscience, New York, USA.
[28]. SAP2000 V-18.2 (2016), Integrated Finite Element Analysis and Design of Structures Basic Analysis, Reference Manual, Berkeley (CA, USA): Computers and Structures Inc.
[29]. Structural Engineers Association of California (SEAOC), (1995), Vision 2000: Performance-Based Seismic Engineering of Buildings, Sacramento, California.
[30]. Uang CM (1991), “Establishing R and Cd Factors for Building Seismic Provisions,” Journal of Structural Engineering, ASCE, 117(1): 19−28.
[31]. UBC (1997) Uniform building code, vol. 2: structural engineering design provisions. International Conference of Building Officials (ICBO), Whittier.
[32]. Vamvatslos, D. &Cornell,A.C. 2001. Incremental dynamic analysis. Earthquake Spectra 20(2), pp. 523-553.
[33]. Zafar, A. (2009) “Response Modification Factor of Reinforced Concrete Moment Resisting Frames in Developing Countries,” Master Thesis, University of Illinois at Urbana-Champaign.
[34]. Zahid M, Robert D and Shahrin F (2013), “An Evaluation of Over-strength Factor of Seismic Designed Low Rise RC Buildings,” rocedia Engineering, 53: 48–51.
[35]. Ziegler MA. Late Permian to Holocene Paleofacies Evolution of the Arabian Plate and its Hydrocarbon Occurrences. GeoArabia 2001; 6 (3) 445-504.

Fayed. M. N, etal. "Seismic Response Modification factors for Multi-Story R.C buildings having Flat Slab System". *IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)*, 17(1), 2020, pp. 01-14.