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Seismic Design Evaluation of Reinforced Concrete Buildings for Near-Source Earthquakes by Using Nonlinear Time History Analyses

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Abstract

Seismic design codes mostly claim that their requirements lead to Life Safety (LS) Performance Level (PL) for buildings. This is while many buildings, designed based on the current codes have shown unacceptable performance, and even have collapsed in some recent earthquakes, particularly near-source events. On this basis, it seems that the code provisions still need further improvement to create sufficient confidence in the engineering community. This study has been conducted to find out how IBC 2009 And ACI 318-2014 codes are effective in providing the LS PL in reinforced concrete multi-story regular buildings with special moment frame lateral load bearing system. For this purpose, a set of multi-story buildings up to 16 stories were considered in the highest seismic hazard zone of Tehran, and were designed based on the codes. Then, a set of near-source three-component accelerograms were employed and scaled according to the code, and a series of nonlinear time history analyses were conducted for all buildings. Roof displacement and acceleration, and base shear forces were calculated, and also the formation trend of plastic hinges and their distribution in the structures were investigated for evaluating the seismic performances. Results show that for some earthquakes the buildings performance exceed LS PL, and even in some cases they reach collapse level.

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Keywords: Multi-story buildings with special moment frames, three-component accelerograms, roof displacement and acceleration, base shear, inter-story drift, plastic hinges, life safety performance level.

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1. Introduction

Most of the current seismic design codes for building systems claim, either explicitly or implicitly, that design of buildings' structures based on their requirements leads to Life Safety (LS) as their minimum Performance Level (PL). This is while some buildings, designed based on the current codes' provisions, and constructed based on high standards, under good supervision, have shown unacceptable PLs, even collapse in some recent earthquakes, particularly near-source events. Also in many cases the extent of damage in the earthquake stricken buildings has been so high, that the demolishing and reconstruction of the building have become inevitable. On this basis, it seems that the code provisions still need improvement to create sufficient confidence in the engineering community.

So far, several studies have been conducted on the adequacy evaluation of seismic design codes' provisions and requirements. Hosseini and Yaghoobi Vayeghan (2000) worked on design verification of an existing 8-story irregular steel building by both push-over and three-dimensional dynamic analyses [1]. With regard to concrete buildings Memari and colleagues (2000) conducted a seismic assessment of an existing 32-story reinforced concrete framed tube building using inelastic dynamic time-history analysis to obtain force and deformation response of the structure subjected to three ground motion records [2]. Fajfar (2000) presented a relatively simple nonlinear method for the seismic analysis of structures (the N2 method) [3]. It combines the pushover analysis of a multi-degree-of-freedom model with the response spectrum analysis of an equivalent single-degree-of-freedom system. Alchalabi (2000) described the application of the Japanese standard method of seismic capacity evaluation of existing reinforced concrete buildings to a hypothetical simple Syrian building, and three level evaluations were made to estimate the building's seismic capacity [4]. Goulet and colleagues (2007) illustrated a state-of-the-art seismic performance assessment through application to a reinforced-concrete moment-frame building designed per 2003 building code provisions [5]. Virote (2008) evaluated the seismic performances of reinforced-concrete buildings by nonlinear static analysis (pushover analysis and modal pushover analysis) and nonlinear time history analysis [6]. Epackachi (2012) studied the linear and nonlinear behavior of one of the tallest RC buildings, a 56-storey structure, located in a high seismic zone in Iran [7]. Masi (2012) evaluated the seismic capacity of some structural models which represent real RC existing buildings designed to gravity loads only, through non-linear dynamic simulations [8]. Thwin (2014) carried out computer aided analysis of twelve storied reinforced-concrete rectangular shape residential building for static and dynamic approach by using ETABS software [9]. Moniri (2014) investigated the results of illustrious characteristics of near-fault ground motions on the seismic response of three reinforced concrete structures (6-Story, 10-Story and 15-Story) [10]. Finally, Yoo (2016) carried out nonlinear dynamic analysis using the PERFORM-3D for small-size pilloti-type RC buildings and assessed their seismic performance [11].

The present study has been conducted to find out how some common seismic design codes are capable in providing LS PL in reinforced concrete multi-story regular buildings with special moment frame lateral load bearing system. For this purpose, a set of multi-story reinforced concrete buildings were considered in the highest seismic hazard zone of Tehran, the capital of Iran, assuming site soil classification of Sc according to IBC-2009. First, the considered buildings were designed based on the regulations of IBC-2009 and ACI 318-2014 code, and it was tried to keep the over-strength as low as possible. In the next step, a set of three-component accelerograms of selected near-source selected earthquakes were employed and scaled according to the code, and a series of nonlinear time history analyses were conducted for all buildings. The response values which were used for evaluating the buildings' seismic performance included roof displacement and acceleration, and base shear forces, all in both main directions. Also, the formation trend of plastic hinges, their corresponding PL as well as their distribution in the buildings' structures were investigated for evaluating the achieved seismic performance.

2. The Considered Buildings

Five 4-, 7-, 10-, 13- and 16-story concrete moment resisting frame buildings with composite floors, were considered, all with the same rectangular plan of 3 by 5 bays, spanning 4.0 to 4.6 meters, and located in the highest seismic hazard zone of Tehran, the capital of Iran, assuming site soil classification of Sc according to IBC-2009. All the considered buildings were designed according to ACI 318-14 and IBC 2009, and it was tried to keep the over-strength as low as possible. Table 1 gives the natural periods of the first three modes of the designed buildings, Figure 1 shows the 3D views of the considered buildings, and Tables 1 to 3 present their specifications.

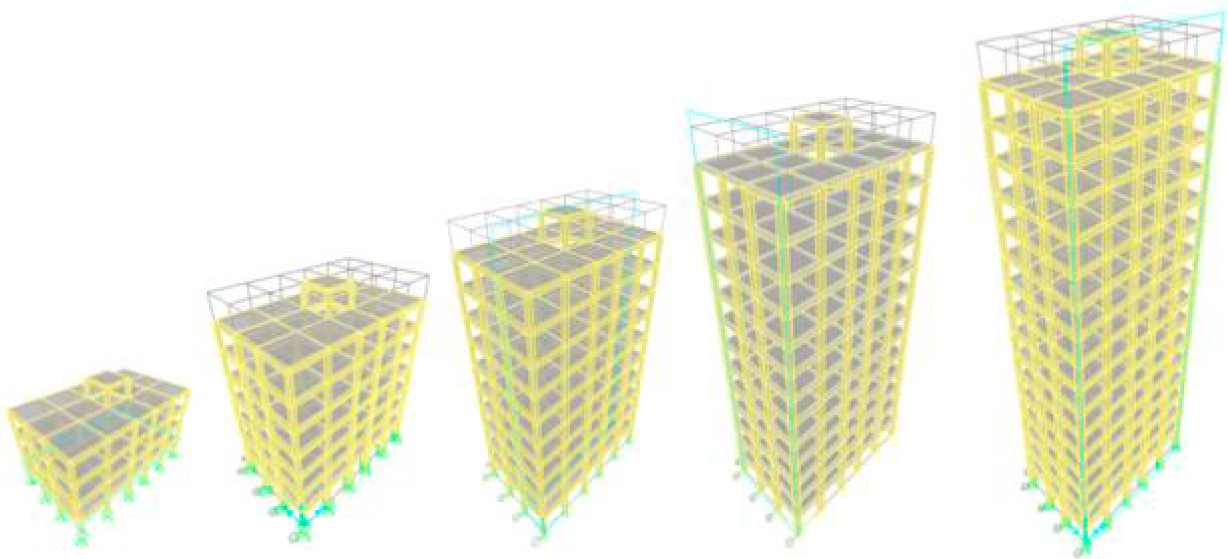

Figure 1. The 3D views of the considered buildings

Table 2. Common specifications of the considered buildings

Story height	3.20 m
Dead load of typical floors	430 kgf/m^2
Dead load of roofs	490 kgf/m^2
Live load of typical floors	200 kgf/m^2
Live load of roofs	150 kgf/m^2
Partitions load	174 kgf/m^2

Table 3. Material properties of the considered buildings

Parameter	Symbol	Confinement steel	Longitudinal steel	Concrete
Modulus of Elasticity	E	2.000E+10 kgf/m^2	2.000E+10kgf/m^2	2.000E+9kgf/m^2
Weight per Unit Volume	w	7850kgf/m^3	7850kgf/m^3	2500 kgf/m^3
Specified Concrete Compressive Strength	f_c'	-	-	250*10^4 kgf/m^2
Minimum Yield Stress	F_y	300*10^5 kgf/m^2	400*10^5kgf/m^2	-
Minimum Tensile Stress	F_u	300*10^5 kgf/m^2	400*10^5kgf/m^2	-
Expected Yield Stress	F_{ye}	32863353 kgf/m^2	43817805kgf/m^2	-
Expected Tensile Stress	F_{ue}	32863353 kgf/m^2	43817805kgf/m^2	-
Poisson	$\mathcal{V}$	0.3	0.3	0.2

Table 1. Un-damped natural periods (sec) of the first three modes of the considered buildings

No. of stories	4	7	10	13	16
Mode 1	0.500	0.732	0.919	1.157	1.369
Mode 2	0.487	0.712	0.890	1.113	1.274
Mode 3	0.450	0.663	0.819	1.002	1.168

It is seen form Table 3 that the first and second periods which are related to lateral modes in the two main directions are close together, as expected. It is also worth mentioning that the third mode of all designed buildings is the torsional mode, and its corresponding period in not much different from the lateral modes periods. The achievable PL of the designed buildings can be obtained by nonlinear time history analysis (NLTHA) as explained in the next section.

3. Seismic Evaluation of the Considered Buildings by NLTHA

To evaluate the adequacy of the employed seismic design code in providing the claimed PL, a set of seven appropriate three-component accelerograms of near-source earthquakes were considered, whose specifications are presented in Table 4, and their response spectra are shown in Figure 2.

Table 4. The selected earthquakes for nonlinear time history analyses and their specifications

Name	PGA (g)	Effective Duration (s)	
		From	to
Northridge	5.5	3.5	11.8
Loma	5.03	3.9	15.4
Landers	2.96	13.1	30.7
Hills	4.28	6.7	20.1
Cape	4.72	5.2	17.1
Kobe	5.66	12	23.5
Hector	5.25	4.5	14.3

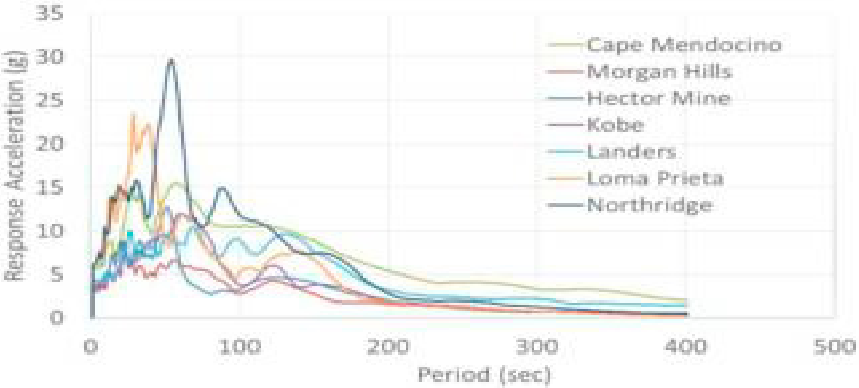

Figure 2. Acceleration response spectra of the selected earthquakes

The selected earthquakes were scaled according to the code, and a series of NLTHA were conducted for all buildings. In these analyses only the elastoplastic behaviour of the beam/column sections were taken into account, and other sources of nonlinearity (e.g. p-delta effects, soil hysteresis, etc) were neglect, and a general viscous damping ratio of 5% were considered. The response values which were used for evaluating the buildings' seismic performance included roof displacement and acceleration, and base shear forces, all in both main directions. Also, the formation trend of plastic hinges, their corresponding PL and their distribution in the buildings' structures were investigated for evaluating the achieved seismic performances. Figures 3 and 4 show some sample response time histories.

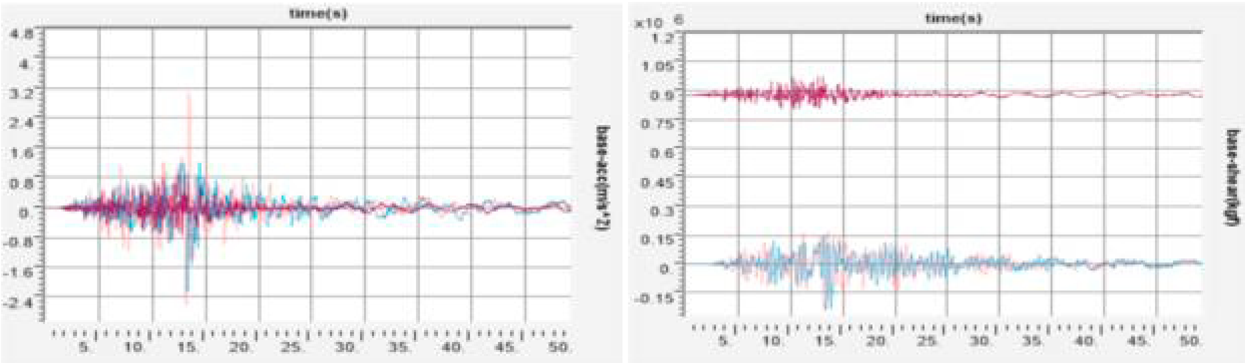

Figure 3. Base accelerations (left) and base forces (right) of the4-story building subjected to Morgan Hill earthquake (pink, blue and red lines stands respectively for x, y and z directions)

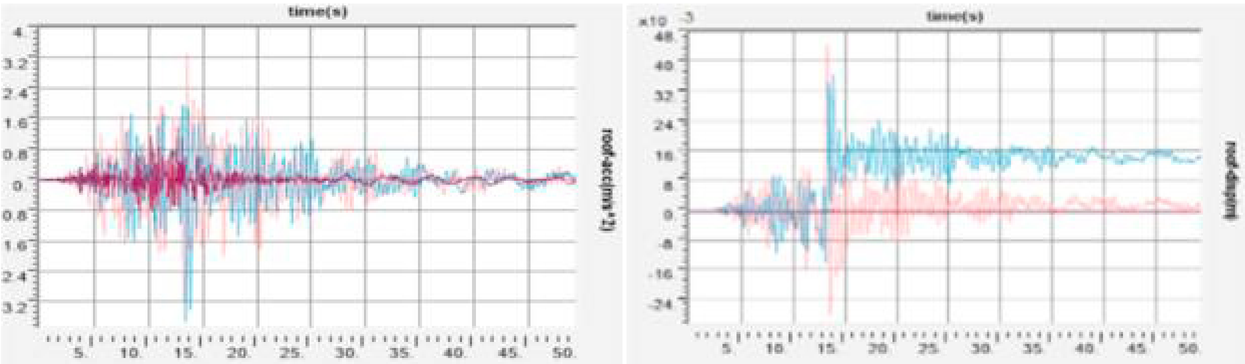

Figure 4. Roof acceleration (left) and displacement (right) histories of 4-story building subjected to Morgan Hill earthquake (pink, blue and red lines stands respectively for x, y and z directions)

It is seen in Figure 3 that the maximum base shear forces are much higher than the code values. Also by looking at Figure 4 one can realize that the amount of roof displacement has exceeded the code limitations. The plastic hinges formed in the structures, seen in Figure 5, also show that the buildings have behaved beyond the expected PL.

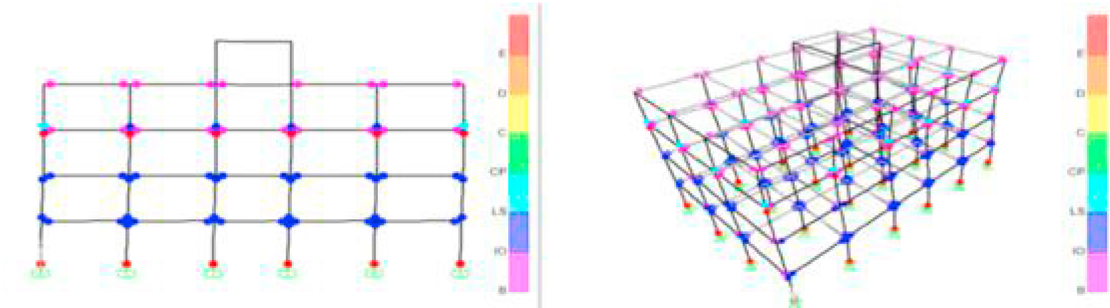

Figure 5. Plastic hinges developed in the 4-story building in subjected to Morgan Hill earthquake

The complete set of maximum responses of the 4-story building are presented in Table 5 (results related to other buildings cannot be presented here because of lack of space).

Table 5. Maximum responses of the 4-story building subjected to the applied earthquakes

Earthquake	**Kobe**	**Northridge**	**Hector Mine**	**Loma Prieta**	**Cape Mendocino**	**Morgan Hill**	**Landers**
PGA(g)	5.66	5.5	5.25	5.03	4.72	4.28	2.96
PGV(mm/s)	3817.116	11674.364	7164.694	6656.664	17923.944	8406.873	8941.246
Base Shear in X Direction(kgf)	3.271e+05	4.380e+05	2.148e+05	3.168e+05	**4.691e+05**	2.442e+05	3.543e+05
	3.381e+05	4.676e+05	1.831e+05	3.033e+05	4.539e+05	2.008e+05	2.499e+05
Base Shear in Y Direction(kgf)	3.643e+05	4.420e+05	2.185e+05	3.472e+05	4.145e+05	2.333e+05	2.920e+05
	3.697e+05	4.358e+05	2.652e+05	3.543e+05	**6.825e+05**	2.628e+05	3.503e+05
Vertical Base Reaction (kgf)	9.667e+05	1.269e+06	9.816e+05	**1.655e+06**	9.474e+05	9.760e+05	1.103e+06
Roof Acceleration-X (m/s^2)	5.107e+00	7.105e+00	3.080e+00	4.767e+00	**9.300e+00**	3.322e+00	4.879e+00
	5.153e+00	6.161e+00	2.614e+00	4.742e+00	6.539e+00	3.317e+00	3.642e+00
Roof Acceleration-Y (m/s^2)	5.547e+00	6.792e+00	3.003e+00	5.390e+00	5.802e+00	3.694e+00	4.236e+00
	5.617e+00	6.564e+00	3.658e+00	5.197e+00	**9.300e+00**	3.817e+00	4.597e+00
Roof Acceleration-Z (m/s^2)	9.578e-01	4.619e+00	1.474e+00	**1.146e+01**	7.686e-01	1.172e+00	2.767e+00
	9.561e-01	4.361e+00	1.484e+00	1.142e+01	7.848e-01	1.176e+00	2.571e+00
Roof Displacement-X (m)	8.666e-02	1.480e-01	3.895e-02	7.995e-02	**2.097e-01**	4.431e-02	8.757e-02
	8.843e-02	1.538e-01	2.988e-02	7.711e-02	1.245e-01	3.446e-02	4.843e-02
Roof Displacement-Y (m)	8.3583-02	**1.342e-01**	2.981e-02	8.000e-02	8.931e-02	3.662e-02	5.022e-02
	8.278e-02	1.197e-01	4.170e-02	7.757e-02	1.946e-01	3.834e-02	7.003e-02
Roof Displacement-Z (m)	7.336e-04	3.334e-04	6.889e-04	**1.005e-03**	7.175e-04	6.960e-4	7.683e-04
	7.363e-04	2.952e-04	7.055e-04	1.003e-03	7.087e-04	7.182e-4	7.898e-04

It can be seen in Table 5 that the maximum value of each specific response, bolded in the table, does not necessarily occurs with one single earthquake. Also the earthquake with maximum PGA value does not necessarily results in the maximum responses in the structure. As other samples of results, roof acceleration and displacement histories of 16-story building are shown in Figure 6.

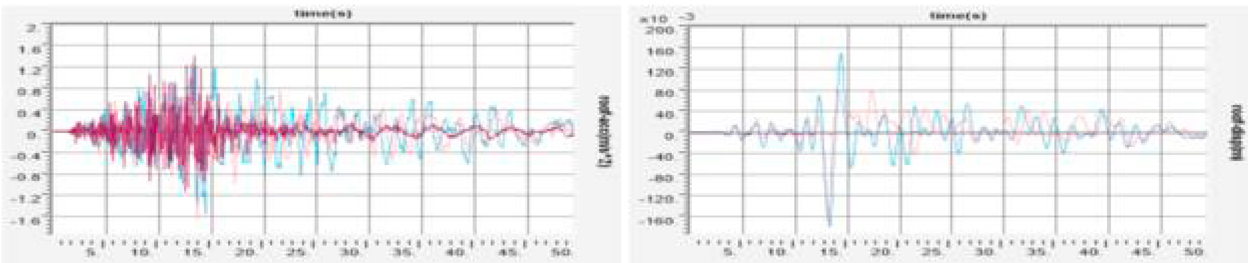

Figure 6. Roof acceleration (left) and displacement (right) histories of 16-story building subjected to Morgan Hill earthquake (pink, blue and red lines stands respectively for x, y and z directions)

More results cannot be presented here because of lack of space, however, the plastic hinges formed in the studied buildings subjected to Morgan Hill earthquake are shown in Figure 7 as samples.

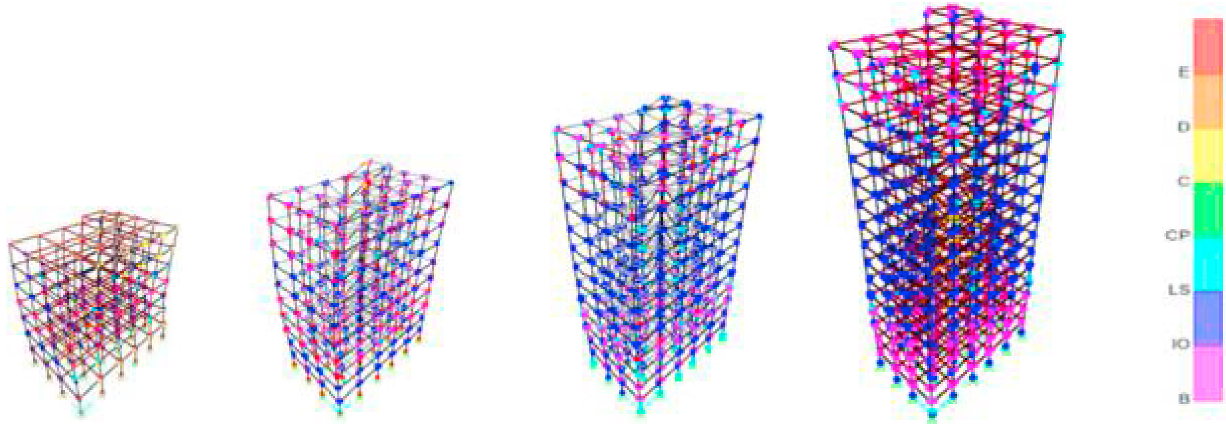

Figure 7. Plastic hinges developed in the considered buildings subjected to Morgan Hill earthquake

It is seen in Figure 7 that in all buildings the formed plastic hinges are beyond the expected life safety performance level.

4. Conclusions

Results show that for some of the employed earthquakes the buildings' performance exceeds the expected performance level, and even in some cases the buildings reach collapse level. This exceedance can be mainly due to the effect of the high intensity of vertical ground excitations. Furthermore, the distribution of plastic hinges in the buildings' structures is not uniform, and they usually concentrate in some specific levels of the buildings, depending on their height and the input earthquake characteristics. On this basis, it can be claimed that the code provisions still need improvement, particularly with regard to the inclusion of the effect of extensive vertical ground motion of near-source earthquakes, to lead to design of buildings which confidently achieve the live safety performance level.

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