Technical Note

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Collapse Safety Margin in Iranian Seismic Design Code: Case Studies of RC Frame Structures

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ABSTRACT

According to the modern seismic design codes, the structural collapse is a catastrophic state which is not acceptable, even under very rare earthquakes. Hence, evaluation of collapse safety margin for structures design based on code requirements is very important. The paper tackles this issue considering RC frame structures designed according to Iranian seismic standard (Standard 2800). Incremental Dynamic Analysis (IDA) is carried out using 22 natural ground motion records. The study includes RC moment resisting frames with 3, 6 and 10 stories considering two types of soil classifications (Type II and III) and two alternatives of ductility levels (intermediate and high), as defined in standard 2800. It is concluded that while all structures on the sites with soil class II demonstrate sufficient margin against collapse, taller structures on soil class III show lower than acceptable collapse margin. It is also noted that the collapse margin is generally reduced with the increased height of the structure.

Keywords:
Collapse Margin Ratio (CMR); Incremental Dynamic Analyses (IDA); RC Frames; Iranian Seismic Standard

1. Introduction

Providing sufficient energy dissipation capacity through plastic deformation is the main goal considered in designing seismic-load-resisting systems for a reduced seismic load. Seismic performance factors (SPF), namely overstrength force reduction factor (R) and deformation amplification factors (C_d), are used to reduce the seismic forces and amplify deformations to arrive at cost-effective and safe designs. On the other hand, restricting the R and C_d values is necessary to prevent excessive inelastic deformations and loss of life, particularly in the event of a major earthquake. Seismic design response factors introduced in seismic codes do not necessarily offer a uniform margin of safety and economic solution considering different seismic regions and the diversity of structural systems, construction practices and quality control. Moreover, modern design codes do not fully address all structural systems currently used in different parts of the world. The capability of these systems to meet the intended seismic design objectives is also not adequately understood [1-2].

To achieve one of the main objectives of the seismic design philosophy, it is important to quantify the margin of safety against structural collapse. Nonetheless due to high level of nonlinearity involved in vicinity of structural collapse the analytical modeling and assessment is complex and demanding, and includes several sources of uncertainties. Although this issue has attracted considerable interest among earthquake engineers and researchers during several past decades, no standard method has been introduced. During the past few years FEMA has published a guideline for quantification of the building Seismic Performance Factors, i.e. response modification factor, overstrength factor and displacement amplification factor [1]. As part of the proposed methodology, one can assess the structural collapse potential [3-6]. The approach includes a combination of incremental nonlinear dynamic analysis (IDA) [7] and suggested criteria based on semi-probabilistic method to evaluate Collapse Margin Ratio (CMR). The methodology has later been extended to consider component rather than whole lateral resisting system [8].

The Methodology achieves the primary, life safety, performance objective by requiring an acceptably low probability of collapse of the seismic force-resisting system for maximum considered earthquake (MCE) ground motions. In general, collapse of a structure would lead to very different numbers of fatalities, depending on the structural system type, the number of building occupants, etc. However, life safety risk (i.e. probability of death or life-threatening injury) is difficult to calculate accurately, due to uncertainty in casualty rates given collapse, and involves even greater uncertainty in assessing the effects of falling hazards in the absence of collapse. Rather than attempting to provide uniform protection of "life safety", the Methodology provides approximate uniform protection against collapse of the structural system. Collapse includes both partial (e.g. single story collapse) and global instability of the seismic force-resisting system, but it does not include local failure of components not governed by the global SPFs (e.g. localized, out-of-plane failure of wall anchorage and potential life-threatening failure of non-structural systems) [9].

The objective of this study is to verify the margin of collapse for the moment resisting structural systems used in the seismic design of Reinforced Concrete (RC) multi-story buildings. This is carried out by using Incremental Dynamic Analysis (IDA) and representative structural characteristics. The study aims to obtain indicative collapse margin ratios for the RC frames designed based on the Iranian seismic standard. All structures are first designed according to Standard 2800 [10] and ACI 318-11 [11]. They are then modeled and analyzed using SAP2000 software and incremental dynamic analysis. The 22 ground motions used in this study are those suggested by FEMA P695. The collapse is assessed for each record and the median value of collapse (S_{CT}) is calculated based on the results for the 22 records. The ratio of median value of collapse to spectral response acceleration at the fundamental period (S_{MT}) is called collapse margin ratio (CMR) [1].

2. FEMA P695 Incremental Dynamic Collapse Analysis

The methodology consists of a probabilistic assessment of collapse risk. It utilizes nonlinear analysis techniques, and explicitly considers uncertainties in ground motion, modeling, design, and test data. The technical approach is a combination of traditional code concepts, advanced nonlinear dynamic analyses, and risk-based assessment techniques. Reliable analysis requires valid ground motions and representative nonlinear models of the seismic-force-resisting system. The main steps of the methodology is illustrated in Figure (1).

Each model is subjected to the predefined ground motions that are "systematically scaled to increasing intensities until median collapse is

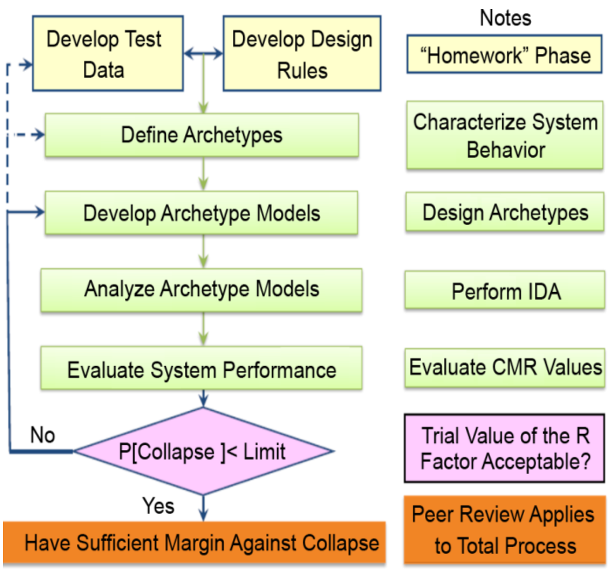

Figure 1. The flowchart for FEMA P695 methodology for collapse assessment [1].

established. Collapse performance is evaluated relative to ground motion intensity associated with the MCE." The methodology defines the collapse level ground motions as, "the intensity that would result in median collapse of the seismic-force-resisting system" [1].

Within the framework of FEMA P695 methodology, two ground motion sets are provided for the nonlinear dynamic analysis used in collapse assessment. One set includes 22 ground motion record pairs from sites located at greater than or equal to 10 km from fault rupture, referred to as the "Far-Field" record set. The other set includes 28 pairs of ground motions recorded at sites less than 10 km from fault rupture, referred to as the "Near-Field" record [1]. The records are scaled in a two-step process: normalizing and scaling. The normalization portion of the process was completed during the development of the record sets. To calculate the collapse capacity by nonlinear time history analysis the amplitude of the ground motion records are scaled based on the fundamental period of vibration for the building under consideration.

System behavior is characterized through the use of structural system archetypes. Archetypes provide a systematic means for characterizing permissible configurations and other significant features of the proposed system. Structural system archetypes are assembled into bins called performance groups, which reflect major divisions or changes in behavior within the archetype design space. The collapse safety of the proposed system is then evaluated for each performance group [12-13].

An incremental nonlinear dynamic analysis of the models subjected to strong ground motions, matched with the design spectrum was carried out to calculate the base shear (V_y). The conversion to spectral coordinates is based on the base shear and the assumption that all the effective seismic weight of the structure (W) participates in the fundamental mode at period (T) [14]. The seismic performance factors are defined in terms of spectral coordinates in Figure (2).

In the following, the main equations to define SPFs are introduced based on the methodology [1] 1.5 times R is shown in the Figure (2) and

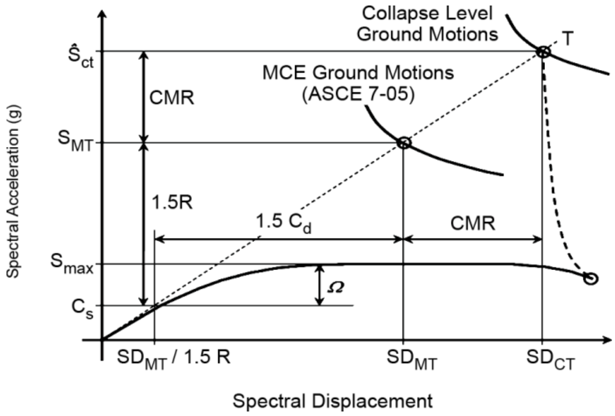

Figure 2. Illustration of Seismic Performance Factors as defined by the methodology [1].

defined as:

$$1.5R = \frac{S_{MT}}{C_S} \quad (1)$$

The collapse margin ratio (CMR) is defined in terms of the ratio of median 5% damped spectral acceleration at the collapse level ground motions (or corresponding displacement, SD_{CT}) to the 5% damped spectral acceleration of the MCE ground motions (S_{MT}) (or displacement, SD_{MT}) [1]. The CMR is calculated as:

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \quad (2)$$

Adjusted collapse margin ratio (ACMR) for each archetype is calculated using spectral shape factors (SSF), which are calculated based on the fundamental period (T) and period-based ductility It is understood that the collapse capacity can be significantly influenced by the frequency content (spectral shape) of the ground motion set. This is attributed to the observed distinctive spectral shape of rare ground motions, such as those corresponds to the MCE, that makes these ground motions to be less demanding than would otherwise be expected based on the shape of standard design spectrum [9]. Therefore, spectral shape factors are introduced as a simplified way of taking this effect into consideration as defined by Eq. (3). Tables 7-1a of FEMA P695 [1] provides the values for SSF.

$$ACMR_i = SSF_i \times CMR_i \quad (3)$$

Adjusted collapse margin ratio (ACMR) is then modified to reflect modeling related, record to record and other sources of collapse uncertainties. Each system is assigned four numerical values based on the following: 1) the confidence in basis of design requirements related to the actual level of behavior to intended results (β_{DR}); 2) the effectiveness of the testing program to quantify properties, behaviors, and failure modes of the system (β_{TD}); 3) the accuracy and robustness of models to represent collapse characteristics (β_{MDL}); and 4) total system collapse uncertainty based on record to record variability (β_{RTR}), which is assigned a set value of 0.4 for the methodology. Since the four component random variables are assumed to be statistically independent, the lognormal standard deviation parameter, β_{TOT}, describing total collapse uncertainty, is given by Eq. (4). Quality ratings for design requirements, test data, and nonlinear models are translated into quantitative values of uncertainty based on the following scale: (A) Superior, β = 0.10; (B) Good, β = 0.20; (C) Fair, β = 0.35; and (D) Poor, β = 0.50. A record to record uncertainty of β_{RTR} = 0.4 is recommended for the index archetype models with a period-based ductility of $\mu_T \geq 3$. Values for total system collapse uncertainty (β_{TOT}) are provided in Tables 7-2a, 7-2b and 7-2c of FEMA P695. Generally, an increase in uncertainty will flatten the curve plotted from IDA. Increased uncertainty in turn increases the probability of collapse at the MCE intensity, SMT, and affects the CMR [1].

$$\beta_{TOT} = \sqrt{\beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2 + \beta_{RTR}^2} \tag{4}$$

The Methodology defines acceptable values of the collapse margin ratio in terms of an acceptably low probability of collapse for MCE ground motions, given uncertainty in the collapse fragility. Calculated values of collapse margin ratio are compared with acceptable values that reflect collapse uncertainty.

Acceptable performance is defined by two basic collapse prevention objectives, requiring that the probability of collapse for MCE ground motions is:

1) Approximately 10%, or less, on average across a performance group, and
2) Approximately 20%, or less, for each index archetype within a performance group.

Consequently, acceptable performance is achieved when the following two criteria is met for each performance group and each index archetype:

- The average value of adjusted collapse margin ratio $(\overline{ACMR_i})$ for each performance group exceeds $ACMR_{10\%}$:

$$\overline{ACMR_i} \geq ACMR_{10\%} \tag{5}$$

- Individual values of adjusted collapse margin ratio ($ACMR_i$) for each index archetype within a performance group exceeds $ACMR_{20\%}$:

$$ACMR_i \geq ACMR_{20\%} \tag{6}$$

Acceptable values of adjusted collapse margin ratios (ACMR10% and ACMR20%) are provided in Table 7-3 of FEMA P695 [1] based on total system collapse uncertainty, β_{TOT}.

3. Nonlinear Modeling of the Archtypes

Within the framework of FEMA P695 methodology for the assessment of seismic performance factored the first step is to gather thorough data about the seismic-force-resisting system. These data includes type of construction materials, system possible configurations, inelastic energy dissipation mechanisms, and intended range of application. Structural system archetypes are developed according to these types of data in order to represent the bounds of proposed seismic-force-resisting system. Structural archetypes provide the basis for preparing a finite number of designs, and then provide a corresponding number of idealized nonlinear models. These models should appropriately represent nonlinear behavior of proposed seismic-force-resisting system [15].

The current study includes RC moment resisting frames with 3, 6 and 10 stories considering two types of soil classifications (Type II and III) and two alternatives of ductility levels (intermediate and high ductilities), according to standard 2800 [10]. The general plan and elevation of the buildings are shown in Figure (3), and it is assumed that the story height for all frames is equal to 3.2 m. The two-dimensional frames considered in this study are considered to be one of the internal frames within the actual three-

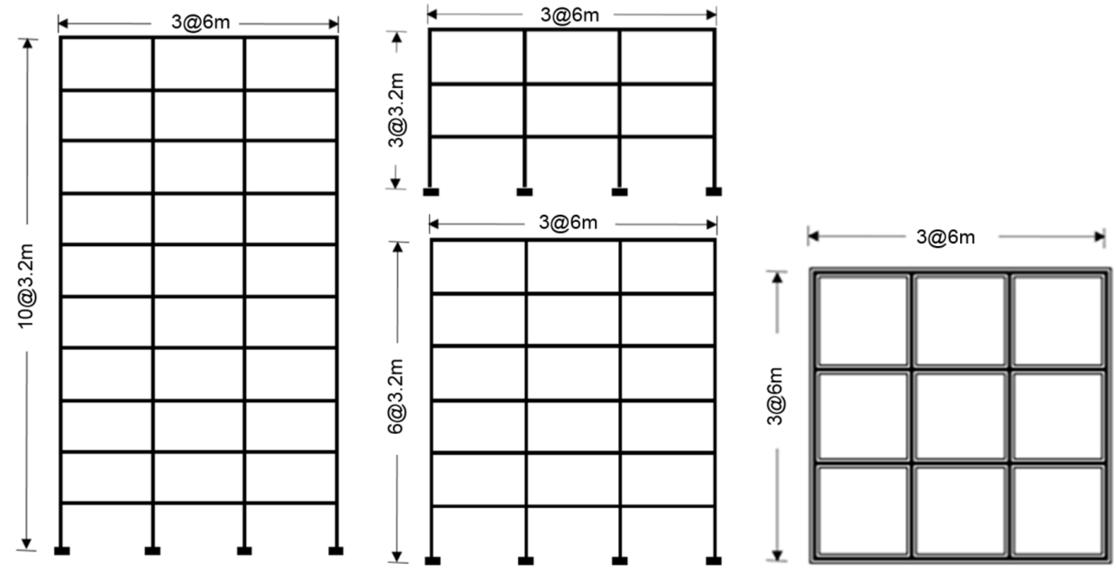

Figure 3. General plan and elevation for 3-, 6- and 10-storey archetypes RC structures.

Table 1. Performance groups for evaluation of archetypes.

Performance Group		Number of Stories	Height	Soil Classifications	Ductility	W (ton)	I	A	R	T (sec)	B	C_S
1	1-A	3	9.6	II	Intermediate	128.7	1	0.3	7	0.38	2.5	0.107
	1-B	6	19.2	II	Intermediate	271.2	1	0.3	7	0.64	2.12	0.09
	1-C	10	32	II	Intermediate	451.8	1	0.3	7	0.942	1.64	0.07
2	2-A	6	19.2	II	High	260	1	0.3	10	0.64	2.12	0.0636
	2-B	10	32	II	High	449.16	1	0.3	10	0.942	1.64	0.049
3	3-A	6	19.2	III	Intermediate	274.7	1	0.3	7	0.64	2.75	0.1178
	4-B	10	32	III	Intermediate	461.4	1	0.3	7	0.942	2.25	0.096
4	4-A	6	19.2	III	High	272.21	1	0.3	7	0.64	2.75	0.0825
	4-B	10	32	III	High	457.4	1	0.3	10	0.942	2.25	0.0675

dimensional building. The design of the frames is based on the initial values for the seismic performance factors according to the Iranian Seismic Standard 2800. All frames are designed according to ACI 318-11. The seismic design specification of the nine index archetype are shown in Table (1). The performance groups considered in this study are somewhat arbitrarily based on the similar soil and ductility classifications. This is acceptable as the aim of the study is to obtain indicative collapse margin ratios for RC concrete moment frames designed based on Iranian seismic standard. For a complete exercise of the FEMA P695 methodology, additional archetypes would be needed.

After designing the aforementioned archetypes, nonlinear dynamic analyses should be performed to investigate system behavior in each case and in every performance group. For this purpose, it is necessary to prepare appropriate nonlinear models of the archetypes. In these analyses, the intensity of the ground motions are continuously scaled up, until the structure is collapsed. At each level of intensity the maximum interstory drift experienced by the structure is plotted against the corresponding spectral acceleration of the record at structural period (IDA curve). The collapse is defined once the dynamic instability occurs and the IDA curve is flatten or the maximum interstory drift ratio becomes larger than 10% [7]. Using the IDA results for all ground motions, the collapse probability can be assessed for the selected archetypes. As indicated above, when the structure collapses its interstory drift rapidly increases (similar to a horizontal line on the figure) [1, 16].

This modeling was carried out using nonlinear modeling features of the SAP2000 software [17]. The nonlinear behavior is modeled using beam and column elements with concentrated plasticity at both ends [18]. To define the characteristics of the plastic hinges, the general backbone curve proposed by ASCE41-06 [19] and its equivalent the Guideline 360 [20] are used, considering the fact that these structures have been designed based on modern seismic design requirements. For the same reason, it is also assumed that the structures are merely collapse in a sideway collapse mechanism and this mechanism is directly simulated by the nonlinear structural modelling. Nonlinear dynamic analyses are conducted under a gravity load combination and input ground motions, which are selected from the far-field record set proposed by FEMA P695. This set consists of 22 pairs of earthquake records. These analyses are utilized to establish the Median Collapse Capacity, SCT, and Collapse Margin Ratio, CMR, for each index archetype model. Median Collapse Capacity is the ground motion intensity in which half of the records within the set cause collapse of an index archetype model. As discussed above this can be established by performing Incremental Dynamic Analyses (IDA).

4. Results

The results of IDA analyses for all archetypes within the four different performance groups (see Table 1) are shown in Figures (4) to (7). The calculated median collapse intensities are also depicted in each figure. As discussed above, the acceptable values of adjusted collapse margin ratio are based on the total system collapse uncertainty and the values of acceptable collapse probabilities. The lower probability of collapse accepted, the larger collapse margin ratio is required to validate the seismic behavior of a system.

Since the study investigates the collapse margin for moment resisting frames, which are designed based on modern seismic design requirements and ACI 318-11 code, the quality ratings for design requirements and test data are assumed to be Good (β_{DR} and β_{TD} = 0.2). Considering the number of archetype models and the analytical software used for the current study, the modelling related collapse uncertainty is assumed to be Fair (β_{MDL} = 0.35). Using these assumptions and also the recommended record to record uncertainty of β_{RTR} = 0.4, the calculated lognormal standard deviation parameter, β_{TOT}, in this study is equal to 0.6. Collapse margin ratios and adjusted values of these ratios for individual archetypes and different performance groups are summarized in Table (2). In addition, the acceptance criteria for each archetype and performance group are also shown in this table in order to be compared with the obtained results. As it is evident from Table (1), acceptable collapse performance is achieved by

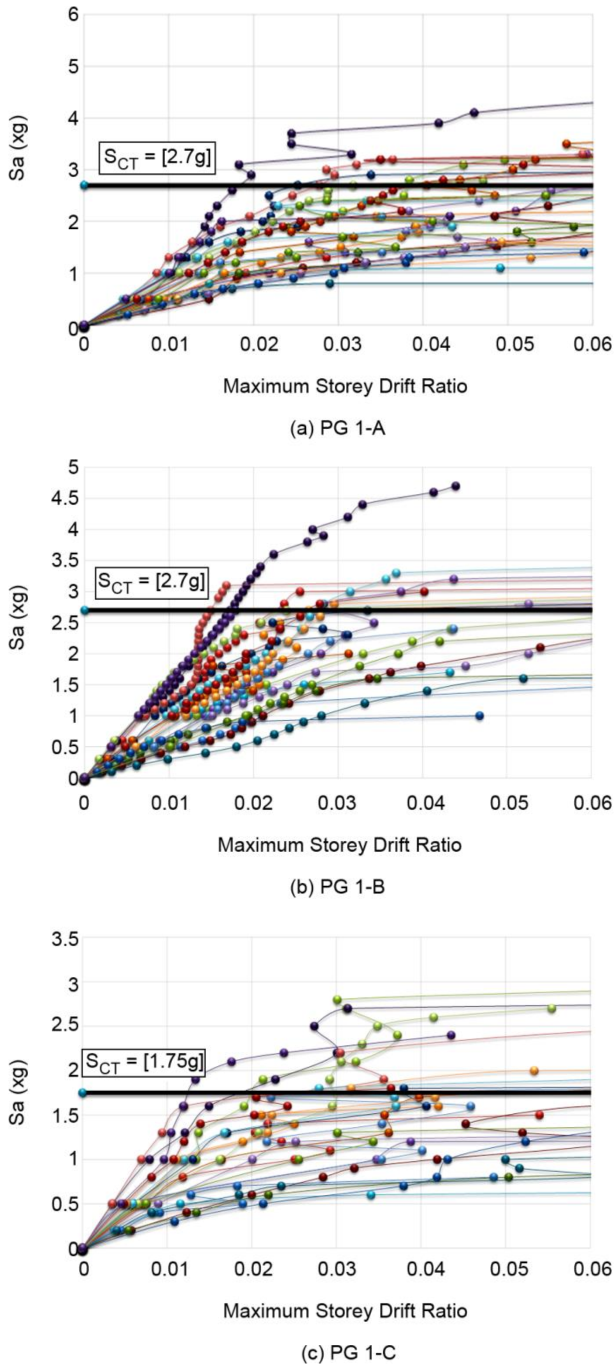

Figure 4. IDA results for archetypes in performance group 1.

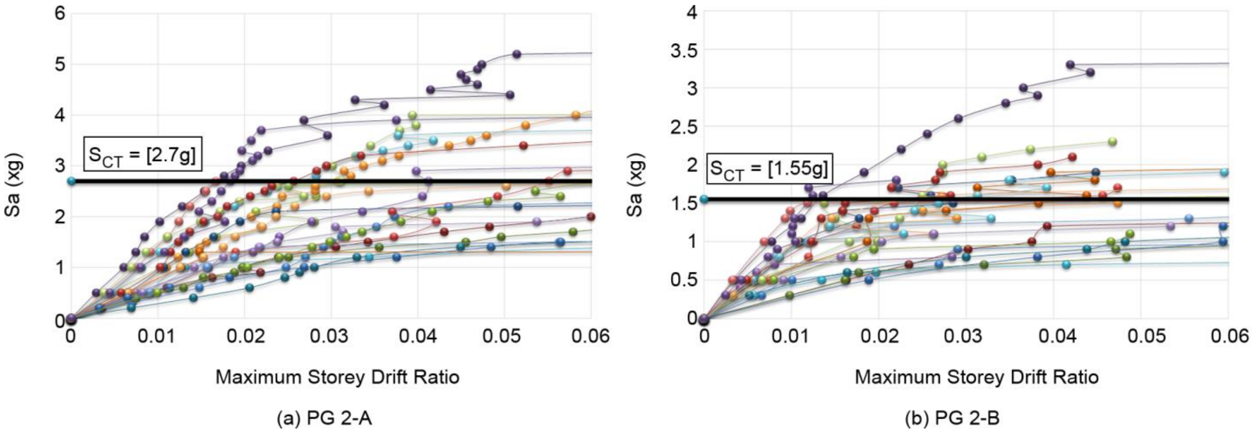

Figure 5. IDA results for archetypes in performance group 2.

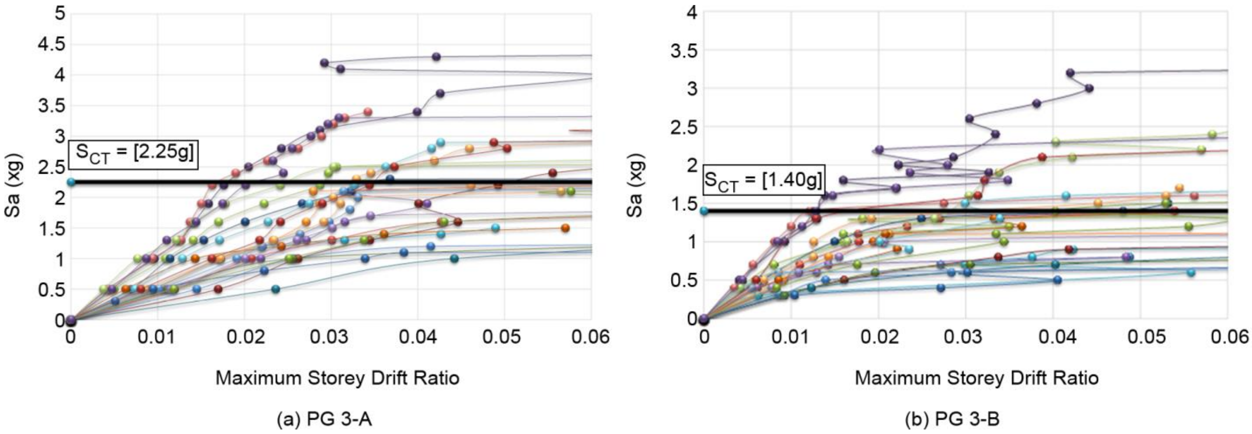

Figure 6. IDA results for archetypes in performance group 3.

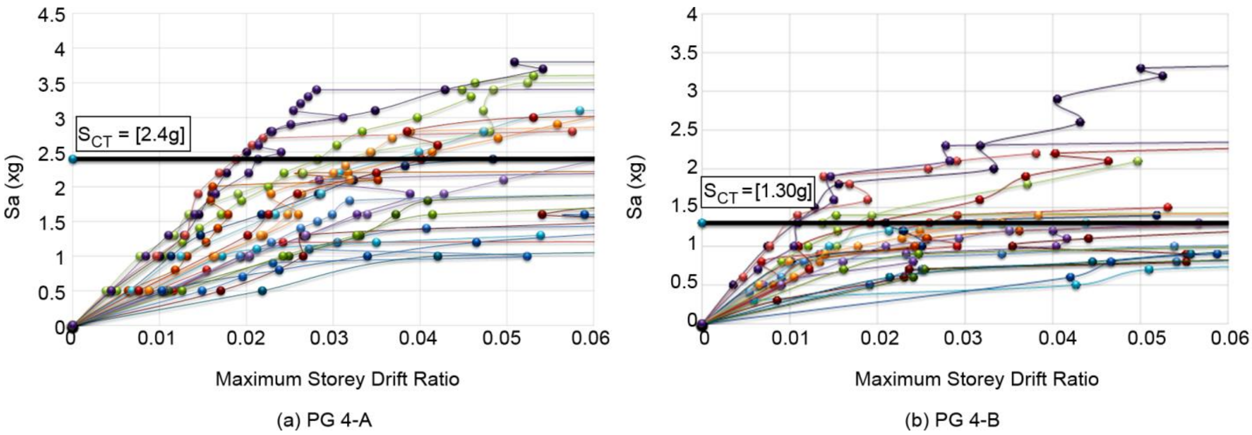

Figure 7. IDA results for archetypes in performance group 4.

all considered index archetypes and performance groups, except performance group 4, which is 10 story building on soil class III.

The average value of Adjusted Collapse Margin Ratio for performance groups 1 and 2 exceeds the acceptable values of Adjusted Collapse Margin Ratio considering 10% acceptable collapse probability, which is equal to 2.16. Again, the individual values of Adjusted Collapse Margin Ratio for the archetypes within the performance groups 1 and 2 exceeds the acceptable values of Adjusted Collapse Margin Ratio considering 20% acceptable collapse probability, which is equal to 1.66. This essentially means that the structures designed at the sites with soil type II, irrespective of their height and ductility level (intermediate or high) have sufficient margin of safety against collapse.

Table 2. Adjusted collapse margin ratio (ACMR).

Performance Group		S_{MT}	$\hat{S}_{CT}$	CMR	SSF	ACMR	B_{TOT}	$\overline{ACMR_i}$	$ACMR_{10\%}$	$ACMR_{20\%}$
1	1-A	1.125	2.7	2.4	1.113	2.67	0.6	2.93	2.16	1.66
	1-B	0.954	2.7	2.83	1.215	3.44	0.6		2.16	1.66
	1-C	0.738	1.75	2.37	1.13	2.68	0.6		2.16	1.66
2	2-A	0.954	2.7	2.83	1.213	3.43	0.6	2.89	2.16	1.66
	2-B	0.738	1.55	2.1	1.118	2.35	0.6		2.16	1.66
3	3-A	1.2375	2.25	1.82	1.218	2.21	0.6	**1.88**	2.16	1.66
	3-B	1.0125	1.4	1.38	1.128	**1.55**	0.6		2.16	1.66
4	4-A	1.2375	2.4	1.94	1.22	2.36	0.6	**1.91**	2.16	1.66
	4-B	1.0125	1.3	1.28	1.134	**1.45**	0.6		2.16	1.66

Performance groups 3 and 4 include 6 and 10 story structures at sites with soil type III and different ductility levels. The results in Table (2) show that while the 6-story structures are individually acceptable in terms of having lower adjusted collapse margin ratios than $ACMR_{20\%}$ = 1.66, both performance groups (3 and 4) fail to satisfy the 10% probability of collapse criterion. This is merely because the 10 story frames have unacceptably low adjusted collapse margin ratio (ACMR < $ACMR_{20\%}$ = 1.66).

Comparing the individual adjusted collapse margin ratios for 6 and 10 story frames, it can be seen that irrespective to their site soil class and ductility level, the 6-story frames show about 30% to 60% higher collapse margin ratio than 10-story frames. This may be interpreted as a reduction in collapse safety margin with structural height. Although only the 3-story frame considered in this study appears not to be in full agreement with this interpretation and demonstrates higher collapse safety margin than corresponding 6-story structure.

The presented results in this paper are based on our best assessment of quantified values to consider the uncertainties (b values). These values are to some extent subjective and debatable. Generally, lower value of total lognormal standard deviation will result in higher collapse safety margins. Therefore, for those structures that are not showing sufficient collapse safety margin, it might be argued that more favorable results may be obtained using different uncertainties. However, this would mean that there should be higher confidences in various parameters. One particular issue relevant to this study is the assumption used for the modelling quality. A brief evaluation of the method assuming a higher modelling quality (β_{MDL} = 0.2) shows that although the collapse margin improves, previous conclusions are not affected and performance groups 3 and 4 are still short of satisfying 10% probability of collapse criterion.

Additionally, it is worth noting that the spectral shape factors (SSFs) used in this study are those suggested by FEMA P695. These SSFs have been derived mainly using the ASCE 7 spectra. The current study uses the spectra from Standard 2800 for the design of the structures and for the scaling of the ground motions. Here, it has been assumed that similar SSFs can be used to adjust the collapse margin ratio for the spectral shape.

5. Conclusions

In this study, the collapse safety margin was assessed for commonly adopted reinforced concrete frame structures that were designed according to Iranian Seismic Design Standard 2800 and ACI 318-11. The study included medium height structures on Class II and III soil sites and intermediate and high ductility levels. The FEMA P695 methodology was used for collapse assessment of the structures through incremental dynamic analysis under a set of 22 far-field ground motions. The acceptance collapse criteria included 10% probability of collapse for an individual archetype and 20% probability of collapse in average for all archetypes in a performance group. The main conclusions are summarized as follows:

- All structures on the sites with type II soil class demonstrate sufficient margin of collapse, irrespective of their height and ductility level.

- 6-story intermediate and special moment frames at the sites with type III soil class also demonstrate sufficient margin of collapse. This is not the case for corresponding 10-story moment frames. Therefore, it seems that the collapse margin is decreased as the height of the structure is increased, i.e. the long period structures are more vulnerable to collapse under severe ground motions when compared to the short period ones.
- Generally, 6-story frames show higher collapse safety margins than corresponding 10-story frames. The adjusted safety margin ratios for 6-story frames are increased within the range of about 30 to 60% as compared with that of 10-story frames.
- Two out of four performance groups considered, all related to type III soil class, failed to satisfy the 20% average probability of collapse criterion, merely due to lack of sufficient margin for the 10-storey frames.
- Based on this study and according to general methodology of FEMA P695, it is also concluded that the assumed response modification factor (behavior factor) of R=7 for intermediate moment frames [10] is acceptable. However, for special moment frames, the R value of 10 appears to be unreliable.

It has to be noted that extensive nonlinear dynamic analyses were carried out in this study to calculate the collapse margin ratio for RC frame structures designed according to the seismic standard 2800 [10]. The definition of performance groups in this study was to some extent arbitrary and the number of archetypes were limited for practical reason. It was aimed to calculate indicative values of collapse margin ratio for these structures. As a result, although the minimum requirements for full implementation of the FEMA P695 procedure is not necessarily satisfied, the conclusions drawn are sufficiently important with regards to seismic performance factors in seismic design codes. Even though to generalize the conclusions of this study, further investigations would be required to include more extensive performance groups.

Last but not least, since the completion of this study the new edition (Edition 4) of Iranian Seismic Design Code (Standard 2800) has been released for public implementation. Some aspects of the code relevant to the collapse capacity evaluation procedure in FEMA P695 have now been modified, such as changes in the values for the behavior factor (R values) for various structural systems, introduction of displacement magnification factor (C_d) and redundancy factor (ρ), explicit specification of overstrength factors (Ω_0), modification of ground motion scaling methodology etc. While the exact evaluation of these changes and their impacts on the structural collapse capacity warrants additional studies, based on the nature of the introduced modifications, the authors believe that the general collapse capacities calculated for *RC* concrete structures in this paper are not expected to be affected, unless for those structures that their design is affected by the newly introduced redundancy factor. For the latter, conceptually higher collapse margin ratios are expected.

References

1. FEMA (2009) *Quantification of Building Seismic Performance Factors*. FEMA P-695, prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

2. May, A. (2011) Assessment of seismic design response factors of concrete wall buildings. *Earthquake Engineering and Engineering Vibrations*, **10**, 115-127.

3. Haselton, C.B., Liel, A.B., and Deierlein, G.G. (2010) Example application of the FEMA P695 (ATC-63) methodology for the collapse performance evaluation of reinforced concrete special moment frame systems. *9th U.S. National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Ontario, Canada.

4. NIST (2010) *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Parameters*. NIST GCR 10-917-8, Gaithersburg, MD.

5. Mahmoudi, M. and Ghasem Abdi, M. (2012) Evaluating response modification factors of TADAS frames. Constructional Steel Research,

162-170.

6. Gogus, A. and Wallace, J. (2015) Seismic safety evaluation of reinforced concrete walls through FEMA P695 methodology. *ASCE J. Structural Engineering*, **141**(10).

7. Vamvatsikos, D. and Cornell, C.A. (2002) Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, **31**, 491-514.

8. FEMA (2011) *Quantification of Building Seismic Performance Factors: Component Equivalency Methodology*. FEMA P-795, prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

9. Kircher, C.A. and Heintz, J.A (2008) ATC-63-recommended methodology for quantification of building system performance and response parameters. *14th World Conference on Earthquake Engineering*, Beijing, China.

10. BHRC (2005) *Iranian Code of Practice for Seismic Resistance Design of Buildings*. Standard No. 2800, Building and Housing Research Center, 3rd Edition.

11. American Concrete Institute (2011) *Building Code Requirements for Structural Concrete and Commentary (ACI 318-11).*

12. Baker, W., Besjak, C., Sarkisian, M., Lee, P., and Doo, C.S. (2010) Proposed methodology to determine seismic performance factors for steel diagrid framed systems. *CTBUH (Council on Tall Buildings and Urban Habitat)*, 13th U.S. Japan Workshop.

13. Donovan, L. and Memari, A. (2011) Determination of Seismic Performance Factors for Structural Insulated Panel Shear Walls Based on FEMA P695 Methodology. PHRC Research Series Report No. 110.

14. FEMA (2004) NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structure. FEMA 450-2/ 2003 Edition. Federal Emergency Management Agency, (Ed.) Washington, D.C, Part 2: Commentary.

15. Farahi, M. and Mofid, M. (2013) On the quantification of seismic performance factors of Chevron Knee Bracings in steel structures. *Engineering Structures*, **46**, 155-164.

16. Zsarnoczay, A. (2013) Influence of plastic mechanism development on the seismic performance of buckling restrained braced frames case study. *Conference of Junior Researchers in Civil Engineering*.

17. CSI (2011) *SAP2000 v-15.0: Integrated Software for Structural Analysis and Design. Computers and Structures Inc.*, Berkeley, California, USA.

18. Tohidian, M. and Manafpour, A.R. (2014) Investigating the behaviour factor of Iranian Seismic Standard 2800 for RC frames based on FEMA P695 procedure. *10th International Congress on Civil Engineering*, University of Tabriz, Tabriz, Iran (in Persian).

19. ASCE (2007) *Seismic Rehabilitation of Existing Buildings*. ASCE Standard ASCE/SEI 41-06, American Society of Civil Engineers, Reston, Virginia.

20. Management and Planning Organization (2007) *Instruction for Seismic Rehabilitation of Existing Buildings, Guideline 360*. Management and Planning Organization, Office of Deputy for Technical Affairs, Technical Criteria Codification & Earthquake Risk Reduction Affairs Bureau, Iran (in Persian).