Uncertainty Quantification in Seismic Collapse Assessment of Iranian Code-Conforming RC Buildings

S.Abbas Hoseini 1, Mohsen Ghaemian 1*, and M.Amin Hariri-Ardebili 2, 3

1Department of Civil Engineering, Sharif University of Technology, Tehran, Iran
2Department of Civil Engineering, University of Colorado, Boulder, USA
3X-Elastica LLC, Boulder, Colorado, USA
*Corresponding author

Abstract

Structural collapse is the main concern in the existing structures which are built in the seismic-prone regions. Therefore, the primary goal of the seismic provisions in building codes is to prevent the global collapse. Iran is located in the Alpine-Himalayan belt, and has experienced some of the most destructive earthquakes in the past century. To evaluate the extent to which the Iranian building code provisions meet this objective, the authors have conducted a detailed assessment of collapse risk on a set of moderate moment resisting reinforced concrete (RC) buildings. While many features might affect the seismic performance of the RC structures, this study considers P-e effects, deterioration in strength and stiffness, and cyclic deterioration in structural components.

Structural assessment is performed using OpenSees platform and the multiple-record incremental dynamic analysis (IDA). Results are presented in terms of the IDA capacity curves and the collapse fragility functions at different seismic hazard levels. Results show that probability of instability increases with height of the buildings. Moreover, the collapse confidence level was evaluated considering the available uncertainties. Assuming a minimum confidence level of 90% for the buildings, the collapse prevention limit state under the 2%/50 hazard level is not satisfied for the 9 and 12 story frames, and they need to re-designed.

Keyword: Collapse; Incremental Dynamic Analysis; Fragility Curve; Confidence Level; Uncertainty

1 Introduction

In earthquake engineering, “structural collapse” refers to the inability of a structural system, or part of it, to maintain the load-carrying capacity under the seismic excitation. Collapse can
be in the local or global levels. The former may occur, for example, if a vertical load-carrying component fails in compression, or if shear transfer is lost between the horizontal and vertical components (i.e., shear failure between flat-slab and a column). One the other hand, the global collapse may occur in several ways: the spread of an initial local failure from element to element may result in cascading or progressive collapse. Incremental collapse occurs if the displacement of an individual story is very large (and the second-order P-Δ effects fully offset the first-order story shear resistance). In either case, replication of the collapse necessitates modeling of deterioration characteristics of the structural components subjected to the cyclic loading (including the P-Δ effects).

During the past earthquakes, several collapses in modern building structures have been reported even though these structures were built in accordance with modern seismic design codes and construction standards. A recent example is the global collapse of the 15 story reinforced concrete (RC) residential Alto-Rio building during the 2010 Maule, Chile earthquake, which had been constructed following the Chilean building codes [1]. Such an observation raises some important questions regarding the capability of the current seismic provisions to provide safety against structural collapse under the extreme seismic forces [2]. Therefore, it is crucial to understand the causes and effects of the structural collapse in order to develop the key documents such as national building codes, regional emergency response plans, and risk management strategies.

In Sec. 1.1 a group of published research articles in seismic collapse assessment of the structures is reviewed. The emphasize will be on the papers related to: 1) collapse in RC structures under dynamic excitation, and 2) advance techniques in uncertainty quantification of the framed structures. Next, the objectives of the present paper is discussed in Sec. 1.2.

### 1.1 Literature Review

#### 1.1.1 Modeling, Analysis and Uncertainties

In general, the research topics in the field of concrete structures can be classified as:

**Experimental studies:** in which a full or scaled down model of the RC frames are tested in the lab. 2D and 3D models are both used for the experiments based on the shake table test, centrifuge, and actuators. Results are usually used for validation of the finite element models.

**Numerical studies:** which contains majority of the existing literature on RC frames. There are two major goals in this part:
• Developing a constitutive model for the nonlinear response of the concrete; its interaction with reinforcement, and the damage simulation. This is an active research field in both micro and macro modeling of the material, structural components and equivalent simplified models. This branch is mainly quantifies the epistemic uncertainty in the material and modeling.

• Investigation on the dynamic response of the RC frames considering different characteristics of the input excitation. Record-to-record (RTR) variability, mainshock-aftershock effect, near- vs. far- field ground motions, directivity, etc. are some of the typical features in this branch. In general, the aim of this group of studies is to quantify the aleatory uncertainty in the demand parameters.

**Risk studies:** in which the raw data from structural and damage analyses are used for the subsequent loss analysis. Finally, these data help to improve the reliability of the structural systems and to risk-based decision-making.

This sub-section is dedicated to review some of the recently published and high-cited papers regarding the modeling, analysis and uncertainty quantification of the RC framed structures. Note that there were many other papers could be listed in our short literature review; however, we kept the list concise.

Kunnath et al. [3] developed an analytical modeling scheme to evaluate the damageability of RC buildings experiencing inelastic behavior under seismic excitation. The numerical model was capable of simulating the ductile moment-resisting frames with shear wall and out-of-plane transverse behavior. The structural model was a combination of concentrated plasticity at the member ends and a distributed flexibility for the element. Baker and Allin Cornell [4] considered the spectral shape in collapse assessment of structures. Lumped plasticity assumption was considered for the beam-column elements and implemented in the OpenSees software [5]. Incremental dynamic analysis (IDA) was then performed to simulate the sideways collapse of the frames.

Goulet et al. [6] evaluated the seismic performance of the RC frames employing the performance-base earthquake engineering (PBEE) framework. Performance was quantified in terms of collapse safety and economic losses. Structural responses include both the ground motion uncertainties and the structural modeling issues. They reported a collapse probability in the range of 2-7% for the buildings subjected to excitations which are scaled to the hazard level equivalent to a 2%/50yr.
Structural modeling is an important issue in seismic response of concrete structures. Haselton and Deierlein [7] evaluated the collapse performance of 30 RC special moment frame buildings ranging from 1 to 20 stories including parametric design variations. The buildings were designed according to the ASCE provisions [8]. Modeling uncertainties were considered in collapse predictions. They found that for 2%/50yr hazard level, the conditional collapse probability ranges from 3-20% with an average value of 11%. The mean annual frequency of collapse, collapse, ranges from 0.7e-4 to 7.0e-4 collapses per year with an average rate of 3.1e-4. This study suggested that the minimum base shear requirement was an important component of ensuring relatively consistent collapse risk for buildings of varying height. Neglecting this requirement has made the taller buildings more vulnerable to collapse.

Liel et al. [9] carried out probabilistic assessment of structural collapse risk through nonlinear time history analysis including the material and modeling uncertainties. Variables such as modeling deformation capacity and post-peak softening response of the components might have a significant influence on the predicted collapse performance [10]. Later, Haselton et al. [11] added the concept of performance-based assessment to enrich the specific design requirements on seismic collapse resistance. Two criteria were considered: 1) the minimum base shear requirement based on ASCE, and 2) the strong-column weak-beam concept based on ACI [12].

Dolsek [13] proposed the extended version of IDA technique in order to include the effects of epistemic uncertainties on the responses. In this method, they first proposed a set of structural models with different combination of material uncertainties. Then, they applied the conventional IDA on each structural model. Finally, integrated all the individual IDA curves in the form of fragility function. They applied this technique on a four story RC frame. Moreover, Celarec and Dolsek [14] conducted a similar research in which the elastic beam-column elements had inelastic rotational hinges at their ends (lumped plasticity). The trilinear moment-rotation relationship in the plastic hinges was the only source of nonlinearity. It was found that the seismic performance of the old three story buildings are mainly controlled by the ultimate rotation of the plastic hinges in the columns, whereas in the case of the contemporary buildings the ultimate rotation of the beams is the controlling parameter.

Based on a series of numerical studies on two RC frame structures, Li et al. [15] claimed that the current tie-force method is inadequate in increasing the progressive collapse resistance (since it does not consider the load redistribution in 3D, dynamic effect, and internal force correction). Then, an improved version of tie-force method was proposed and its reliability on the RC frames was verified. In a comprehensive paper, Kam et al. [16]
described the observations of damage to RC buildings in the 22 February 2011 Christchurch earthquake. Damage statistics and typical damage patterns were presented for various configurations and lateral resisting systems. They emphasized on the fact that some aspects of the seismic design should be improved.

Fragiadakis et al. [17] studied the applicability of nonlinear static procedures to estimate the seismic demands of the typical moment-resisting RC frames. They compared different nonlinear static procedures and validated against the nonlinear response history analysis. They quantified the degree to which the nonlinear static methods can characterize the local and global demand parameters. Lu et al. [18] investigated the collapse resistance of two existing RC high-rise buildings of 18 and 20 story frame-core tube systems. They used the finite element technique with fiber beam element model, multilayer shell model, and elemental deactivation technique to predict the collapse process.

Raghunandan and Liel [19] studied the effect of ground motion on the collapse of 2D RC frame structures. The structural models include three bay frames with different heights. They used the OpenSees software and the lumped plasticity beam-column elements [20] in conjunction with inelastic joint shear springs. IDA technique was employed to perform the numerous nonlinear simulations. Furthermore, Raghunandan et al. [21] quantified the aftershock vulnerability of four modern ductile RC framed buildings in California by conducting IDA on nonlinear analytical models. Collapse and damage fragility curves were subsequently derived. They reported that if the building is extensively damaged in the mainshock, there is a significant reduction in its collapse capacity in the aftershock. In addition, Riahi et al. [22] compared the seismic structural response of a set of RC moment resisting frames under excitation of real accelerograms and ground motions that are spectrally matched to a target spectrum. The matching process was conducted in the time domain, and the ASCE 7-05 spectrum was used as the target spectrum.

Sattar and Liel [23] studied the impact of three collapse indicators (i.e. column-to-beam strength ratio, ratio of shear strengths in adjacent stories, and ratio of column flexural-to-shear strength) on the collapse performance of RC moment frames. They also quantified the relation between the collapse performance and collapse indicators for varying column and frame characteristics. They reported that for the buildings with flexurally-dominated columns, the significance of the deficiency is important, while for the shear-critical columns, the location and significance of the deficiency are covering parameters. Some other researchers focused on interaction between the infill walls and the concrete frames [24] and [25]. They studied the collapse mechanism of the retrofitted frames compared to the bare-frame. The
sustainability of the RC framed structures studied in the context of PBEE by Haghpanah et al. [26]. Moreover, Tafakori et al. [27] investigated the dominant collapse mechanism of RC frames accounting for the modeling uncertainty and the RTR variability based on IDA.

1.1.2 National and International Codes

One of the final goals in all the research studies is to propose a model and/or criteria to be used by practitioners. There are several studies in which the seismic performance of RC buildings designed according to the current design codes is investigated. Kueht and Hueste [28] and Kim and Kim [29] studied the seismic performance of a RC building designed based on the 2003 International Building Code (IBC). Panagiotakos and Fardis [30] validated the seismic performance of the RC buildings based on the Eurocode 8. Kotronis et al. [31] developed a simplified approach for RC walls which was a combination of the Bernoulli multi-layered beam elements, the concept of damage mechanics and plasticity. Then, it is used to simulate two RC wall specimens designed based on the French code PS92 and the Eurocode 8. Sadjadi et al. [32] investigated different aspects of the RC framed structures designed based on the National Building Code of Canada. Similar researches were reported by Tena-Colunga et al. [33] for the buildings designed according to the Mexico Federal District Code. Moreover, Mehanny and El Howary [34] and El Howary and Mehanny [35] focused on the buildings designed based on the Egyptian seismic code. Finally, Duan and Hueste [36] investigated the seismic performance of a multi-story RC frame building designed according to the provisions of the current Chinese seismic code, GB50011-2010. Astriana et al. [37] utilized fragility curves in order to compare the seismic performance of moment-resisting frames and frame-wall systems. Results were compared based on the recommended methods in HAZUS-MH MR5 and ATC-40.

A review of previous studies shows the lack of a comprehensive study on the seismic performance of RC buildings designed according to Iranian National regulations. Iran is a seismic-prone country located on the Alpine-Himalayan orogenic belt and has experienced several distributive earthquakes in the past decades. According to Saloor and Salari [38], there was over 100,000 loss of life as a result of earthquakes in Iran over the past 35 years. Figure 1 illustrates the level of seismicity in Iran during the period 2006-2015. Some of the major past earthquakes were: the 1996 $M_w=6.1$ earthquake in Ardabil, the 2003 $M_w=6.6$ earthquake in Bam, the 2012 $M_w=6.4$ earthquake in Ahar, and the 2017 $M_w=7.3$ earthquake in Kermanshah. Furthermore, the seismic site effect of the site of Tehran can be found in [39] and [40].
1.2 Objectives

Statistically, most of the concrete buildings in Iran have been designed and built with a moment-resisting frame system. Majority of the past studies that have examined the vulnerability of concrete buildings have been devoted to short buildings with a special moment-resisting frame system or frame-wall system. However, a large number of short-and mid-rise buildings are designed at the level of moderate ductility according to the ninth topic of the national building regulations.

Therefore, in this research, the effect of the structural height and the average ductility capacity level on the collapse prevention (CP) and global instability (GI) performance levels are investigated. To consider the height effect, five type of buildings are designed with number of stories from 3, 6, 9, and 12 to 15. The major objectives of the study are summarized as follows:

- Utilizing an effective procedure to identify collapse criteria of framed structures from their dynamic instability, i.e., the loss of the ability to sustain the gravity loads.
- Develop a probabilistic approach of collapse assessment based on collapse limit state (LS) to promote reliable probabilistic evaluation of structural collapse.
- Provide fragility functions through systematic treatment of uncertainties in seismic capacity, demand, and structural models for integration within PBEE framework.
- Evaluating the decadency of the designed structures for a 2%/50yr event in different locations of Tehran.

2 Case Studies

Five buildings with 3, 6, 9, 12 and 15 stories were designed according to Iranian concrete regulation (a.k.a. ABA) and Iranian seismic design regulation (a.k.a. Standard 2008), and further controlled based on ACI 318-11 criteria \[12\]. All the structures were designed as residential building. All the 3D modeling and design process are conducted using ETABS software \[41\]. The plan view of the buildings are shown in Figure 2, where the height of stories are 3.20 m. It is assumed that the buildings are located in the area with a high relative seismic risk, with soil type II. Ductility of the structures is assumed to be moderate, R = 7. Dead and live loads are assumed to be 650 and 200 kg/m\(^2\), respectively. The compressive strength of the concrete is 240 kg/cm\(^2\), the main and enclosing reinforcement yield resistance are 4,000 and 3,000 kg/cm\(^2\), respectively. The ceiling system is in the form of joists and a block, and joists direction is shown in Figure 2. Moreover, the relative interstory drift for the
buildings (as a result of linear seismic design) and the allowable one per Iranian seismic design regulation are shown in Figure 3. As seen, the displacements meet the requirement.

To evaluate the nonlinear behavior of the designed frames, OpenSees software with plastic hinge modeling method have been used [5]. In this method, the nonlinear behavior of the elements is defined by plastic hinges and is devoted to the element in the middle of the so-called plastic region (which is one of the nonlinearity sources in the element). The model used for plastic hinge is shown in Figure 4(a).

These relationships are suitable for modeling the elements of designed buildings. The model used by Haselton et al. [42] for nonlinear behavior of beam and column elements is the peak oriented model presented by Ibarra et al. [20]. This model is plotted in Figure 4(a). This hysteresis model has the ability to take into account the deterioration of stiffness and resistance in different behavioral branches. In Figure 4(b), the observed hysteresis behavior was compared with the results of a laboratory test carried out at the University of Tehran [43]. As seen, there is a good consistency between the numerical and experimental models.

In this paper, the above-mentioned parameters are used to model the beam-column dynamic behavior. For comparison purposes, two types of stiffness models were used. One is based on the model used in Haselton et al. [42], and the other one follows the recommendations in the ASCE/SEI 7 [8] regulation. The initial secant stiffness is in the following form based on ASCE/SEI 7 [8]:

\[
\left( \frac{EI_x}{EI_y} \right)_{ASCE} = 0.2 + \left[ \frac{P}{A g f_c} \right] \geq 0.3
\]  

(1)

The values given for the parameter of ratio of the effective stiffness, \( K_e \), of the uncoupled plastic hinge to the un-cracked section stiffness, \( K_g \), in Haselton et al. [42] and ASCE/SEI 7 [8] for a few beam and column of a nine story building are given in Table 1. As seen, the stiffness of the columns using the relationship provided by ASCE/SEI 7 [8], which depends on the axial load of the column, is greater than the average results of Haselton et al. [42].

3 Nonlinear Static Analysis

A group of static nonlinear analyses have been performed on the built-in model, in order to determine the relationship between the base shear and roof drift. This helps to quantify the sensitivity of different modeling parameters. Two lateral loading patterns are used for static nonlinear analyses of the three story frame: 1) triangular and 2) uniform. For other frames, the
loading patterns include: 1) uniform distributions, and 2) spectral analysis-based shear distribution. The resulted curves in the form of base shear vs. the roof drift are shown in Figure 5, known also as pushover or capacity curves.

In general, the capacity curves resulted from different modeling techniques are similar. The initial stiffness of the curves correspond to the modeling with parameters recommended in ASCE/SEI 7 [8] is slightly higher than Haselton et al. [42]. Using these plots, a drift corresponds to the instability is determined. Based on FEMA P695 [44], it is equal to the drift corresponds to 80% of the maximum base shear (i.e., the point where the resistance is dropped by 20%). Table 2, illustrates the drift values for initiation of the instability with the proposed parameters in Haselton et al. [42]. The results show that by increasing the stories, the instability drift is decreased. It can be mainly attributed to the geometric effects of the vertical loads.

4 Incremental Dynamic Analysis

In general, there are many narrow- and wide-range analysis techniques can be used for performance evaluation of structural and infra-structural system [45, 46]. In this paper, the seismic performance of the frames are evaluated based on IDA [47, 48]. This technique needs a group of ground motion records (usually 20 to 40). According to the recommendations by Shome [49], 20 ground motion records have been selected to investigate the RTR variability in the demand parameters. Moment magnitude, $M_w$, varies from 6.5 to 7.5, and they were selected based on the seismic hazard analysis on Tehran Province, Iran [50, 51, 52, 53]. These ground motions are listed in Table 3, and the elastic response spectra are shown in Figure 6.

In IDA method, the intensity of the selected ground motions are increased incrementally until the structure losses its dynamic stability (either due to collapse or numerical un-convergence). Therefore, 10 series of full IDA analyses are performed (5 structural model and 2 types of stiffness assumptions). The associated single IDA curves along with the mean, and standard deviation are shown in Figure 7 for the effective stiffness based on relationships in [42].

Two limit states (LSs) are considered, i.e., collapse prevention (CP), and global instability (GI). According to the IM-based rule [20], the last point (with the highest IM) on the IDA curve with a tangent slope 20% of the elastic part is defined as the CP point. This point should has the maximum interstory drift equal or less than 10% of the maximum drift, $\theta_{max} =$
10%. If not, the point associated with $\theta_{\text{max}} = 10\%$ is chosen as the CP point. Furthermore, the GI corresponds to the flat part of the IDA curve.

5 Fragility Curves and Damage Assessment

A fragility function quantifies the probability of exceeding a particular damage level (i.e., LS, or structural collapse) as a function of ground motion IM [54]. The concept of a fragility function in earthquake engineering goes back at least to 1980s, where Kennedy et al. [55] defined a fragility function as a probabilistic relationship between frequency of failure (in this work a component of nuclear power plant) and environmental excitation (these authors speak exclusively of earthquakes and PGA) [56].

The distinction among three fragility functions (i.e., empirical, analytical, and expert opinion) was discussed by Porter [57]. Empirical fragility curves are derived from post-earthquake damage data [58, 59]. Analytical fragility curves are based on numerical transient structural analysis [60, 61]. Moreover, the heuristic fragility curves are developed based on the expert opinion [62].

Usually, a log-normal cumulative distribution function (CDF) is adopted to fit a fragility function [61]. The collapse fragility curve is quantified as:

$$P[C|\text{IM} = im] = \phi \left[ \frac{\ln(im) - \ln(\eta)}{\beta_{\text{RTR}}} \right]$$

(2)

Where $P[C|\text{IM} = im]$ is the probability that the structure will collapse under a ground motion with an intensity level, $im$; $C$ refers to structural collapse; $\phi(.)$ is the standard normal cumulative distribution function; $\beta_{\text{RTR}}$ the logarithmic standard deviation (also called dispersion) due to RTR variability, and $\eta$ median of the fragility function [56].

Based on the above discussion, the fragility curves were derived for the CP and GI limit states, as shown in Figure 8. The spectral acceleration at the structure's first mode is taken as IM parameter. It can be found that in most cases, the fragility curves obtained from stiffness modeling with the experimental parameters [42] have higher probability than those obtained from [8]. The other institutive observation is that probability of exceedance of CP limit state is higher than GI.

In order to investigate the impact of structural height on the probability of exceedance, the fragility curves for all the frames with the proposed modeling parameters of [42] are shown in Figure 9. It can be seen that, increasing the height of structure, reduces the required IM parameters to meet a particular probability of exceedance. The reason can be attributed to
the fact that increasing the structural height, increases the fundamental vibration period. Subsequently, the spectral acceleration of the selected records are decreased at the fundamental vibration period. This rule is feasible for the frames with 3 to 12 stories. However, the fragility curve of 15 story frame is lower than 12 story one. In this case, increasing the fundamental vibration period, increases the spectral acceleration as well. Thus, the frequency content of the selected ground motion records play an important role in performance assessment procedure.

In general four type of uncertainties are considered in this paper:

- Ground motion record-to-record variability, $\beta_{RTR}$, which is directly obtained from the IDA curves.
- Modeling uncertainties, $\beta_{MDL}$, is assumed to be 0.2 considering the fact that a comprehensive models of the frames was made [63].
- Designing requirements’ uncertainties, $\beta_{DR}$, is assumed to be 0.1, because the studied buildings were designed according to code requirements.
- Experimental test data's uncertainties, $\beta_{TD}$, is assumed to be 0.2, given the fact that a lot of experimental tests were used to extract the force-displacement relation.

Finally, the total uncertainty can be calculated using the SRSS rule [64]:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{MDL}^2 + \beta_{TD}^2 + \beta_{DR}^2}$$ (3)

Note that there are two major assumptions in Eq. 3: 1) all the uncertainty sources are independent, and 2) median of the fragility curve remains constant while the dispersion is altered. Some of the recent studies have shown that the material/modeling uncertainty not only increases dispersion but also affects the median response [9, 65] to some extent. However, for all the practical purposes, one can combine the uncertainties using Eq. 3 [66, 67]. Figure 10 compares the collapse fragility curves for the 9 story frame using $\beta_{RTR}$ and $\beta_{TOT}$.

Results of this study can be used to evaluate the structural damage when the frames are subjected to the maximum credible earthquakes (MCE). The spectral acceleration for all the buildings are extracted from the seismic hazard maps of Tehran with a return period of 2,475 years, Figure 11 [68]. Two different locations are used with different seismicity conditions. Both are located on soil type II; however, with PGA equal to 0.35 and 0.45 g, respectively.

Using the developed fragility curves, the probability of exceedance of CP and GI limit states were determined, Figure 12. It can be seen that, the potential instability of 3 and 6 story
frames is less than other ones. The 9 story frame has the worst condition. Two major observations are: 1) increasing the PGA level, increases the probability of exceedance of LSs considerably, and 2) in both PGA levels, the probability of exceedance of CP limit state is higher than GI. Moreover, one should notice the differences between two types of assumptions in modeling the frame stiffness. For some of the frames subjected to a particular seismic hazard, using the alternative stiffness modeling, may increase to decrease the probability of exceedance up to two times.

6 Reliability Analysis

Usually several factors are contributing in performance assessment and risk analysis of engineering structures. These probabilistic approaches are used to determine the earth-quake ground motions and the material uncertainties. Therefore, a statistical framework is required to compare the reliability of the seismic responses.

In this paper, the confidence factor approach is used to estimate the collapse confidence levels. The fundamental concept of this approach can be found in Wen and Foutch [69], Hamburger [70], Jalayer [71], which was adopted by different researcher in various steel and concrete structures [72, 73]. The target threshold is normally set to be 90% confidence level for the CP limit state in the earthquake with a return period of 2,475 years. It is noteworthy that the relationship and the uncertainties used in this section are basically proposed for the steel structures; however, due to the lack of suitable data for concrete structures, the similar values and relationships are used.

The acceptance criterion is based on a confidence factor, \( \lambda \), which is used to determine the confidence level. This factor is the ratio of the factored demand over the capacity, and can be expressed as:

\[
\lambda = \frac{\gamma \gamma_a D}{\varphi C}
\]  

(4)

Where \( D \) presents the estimate of median drift demand, \( C \) is the estimate of median drift capacity, \( \varphi \) is resistance factor, \( \gamma \) is demand factor, and \( \gamma_a \) is analysis demand factor.

The resistance factor is computed as:

\[
\varphi = \varphi_{RC} \varphi_{UC}
\]  

(5)

\[
\varphi_{RC} = e^{-k \beta_{RC}^2/2b}
\]  

(6)

\[
\varphi_{UC} = e^{-k \beta_{UC}^2/2b}
\]  

(7)
where $\varphi_{RC}$ is the contribution to $\varphi$ from ground motion variability; $\varphi_{UC}$ is the contribution to $\varphi$ from uncertainties in measured component capacity; $\beta_{RC}$ can be interpreted in global (i.e., standard deviation of the natural logs of the drift capacities from IDA analysis and independent from the demand uncertainty) and local (i.e., test variability in rotation capacity for the SAC project and set to 0.20 based on test results) levels; $\beta_{UC}$ is the standard deviation of the natural logs of the drift capacities derived from experiment; $b$ is assumed to be 1.0 for this application; and $k$ presents the slope of the hazard curve calculated from Seismic hazard analysis.

The parameter $k$ is a function of the hazard level, location, and vibration period. The hazard curve is a plot of probability of exceedance of a spectral ordinate versus the spectral amplitude for a given period and is usually plotted on a logarithmic scale. In functional form, it is expressed as:

$$H_{Si}(S_i) = k_0 S_i^{-k}$$  \hspace{1cm} (8)

The value of $k$ can be obtained by re-arranging the above equation with the two spectral values for any two hazard levels. In this study, 2%/50 years and 10%/50 years hazard levels are used to calculate the slope of the curve, $k$. The equation is in the form:

$$k = \ln\left(\frac{H_{Sa}(S_{a10%/50})}{H_{Sa}(S_{a2%/50})}\right)$$  \hspace{1cm} \ln\left(\frac{S_{a2%/50}}{S_{a10%/50}}\right)$$  \hspace{1cm} (9)

Where $S_{a2%/50}$ and $S_{a10%/50}$ are the spectral amplitude for 2% and 10% in 50 years, respectively. $H_{Sa}(S_{a2%/50})$ and $H_{Sa}(S_{a10%/50})$ present the probability of exceedance for 2% and 10% in 50 years; $i$ is the period of interest (0.3 sec for $S_s$ and 1.0 sec for $S_1$).

The capacities determined by testing are subjected to uncertainties. In some cases, it is likely that there will be not enough specimens to determine a reliable estimate of $\beta_{UC}$. In this case, it is recommended that the test data from similar specimens should be used along with the new test results. For SAC studies, $\beta_{UC} = 0.25$ was determined to be a good representative value. Since there is no established value for the concrete structures, we assume the same number as well.

The demand factor, $\gamma$, is calculated as:

$$\gamma = e^{k\beta_{RD}^2/2b}$$  \hspace{1cm} $\beta_{RD} = \sqrt{\sum \beta_i^2}$ \hspace{1cm} (10)

Where $\beta_i^2$ is the variance of the natural log of the drifts for each element of uncertainty.
The $\beta$ values for each source of uncertainty and randomness are as follows: $\beta_{acc}$, ground motions (demand drifts), and $\beta_{or}$, orientation. The orientation factor applies only for the near-fault California sites where known faults are mapped, so that away from the near-fault sites $\beta_{RD} = \beta_{acc}$. For this case, $\beta_{acc}$ is the standard deviation of the log of the maximum story drifts calculated for each of the 20 representative ground motions.

The demand factor $\gamma_a$ is based on the uncertainties related to the determination of demand, $D$, and is calculated as:

$$\gamma_a = e^{k\beta_a^2/2b}$$

(11)

Where $\beta_a$ term is equal to the square root of the sum of the squares of the $\beta$ values determined from each of four uncertainty sources: 1) $\beta_{NTH}$ is associated with uncertainties in the nonlinear time history analysis procedure, 2) $\beta_{damping}$ is associated with uncertainty in estimating the damping value of the structure, 3) $\beta_{live}$ is associated with the uncertainty in applied live load, and 4) $\beta_{material}$ is associated with uncertainty in material property. Only the $\beta_{NTH}$ and $\beta_{damping}$ are large enough to be included in reinforced concrete moment-resisting buildings [74].

The $\beta_{UT}$ term is a function of the all uncertainty components. Therefore, it comprises uncertainties associated with the demand and capacity, but does not account for their stochastic nature. It only accounts for the $\beta_U$ (from capacity) and $\beta_a$ (from demand). Therefore, $\beta_{UT} = \sqrt{\beta_C^2 + \beta_a^2}$.

The confidence factor, $\lambda$, depends on the slope of the hazard curve, $k$, and the uncertainty associated with the natural log of the drifts:

$$\lambda = e^{-\beta_{UT}(K_x - k\beta_{UT}/2b)}$$

(12)

Where $\beta_{UT}^2 = \sum \beta_i^2$ ($\beta_i$ are the uncertainties in the demand and capacity), and $K_x$ is the standard Gaussian variate associated with probability $x$ of not being exceeded. One can rewrite the equation as:

$$K_x = \frac{1}{\beta_{UT}}\left[-\ln \lambda + \frac{k}{2b}\beta_{UT}^2\right]$$

(13)

Finally, $K_x$ can be presented in a probabilistic form as:

$$P(K_x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{K_x} e^{-x^2/2} dx$$

(14)
Using these equations, the confidence levels in CP and GI are calculated for all the buildings keeping in mind two locations with PGA equal to 0.35 and 0.45 g. Detailed calculations are reported in Tables 4 to 7. It can be found that the frames with higher probability of collapse or instability have a lower confidence level. Also, it is observed that the collapse confidence level of the frames modeled with the proposed parameters in [42] is slightly lower than those modeled based on reference [8]. In the case of 12 story frame, these results are vice versa. Tables 4 and 5 show that for the region with PGA = 0.35 g, the confidence level for CP and GI is higher than 90%. On the other hand, Tables 6 and 7 show that for the region with PGA = 0.45 g, the confidence level in 9 and 12 story frames is low enough to re-design those frames again with higher resistance. In general, it is conventional to consider the 90% confidence level as a threshold for the CP [66]. In the present study, we also accept the 90% confidence level for the GI limit state in the earthquake events with the return period of 2,475 years. Thus, for the regions with PGA = 0.35g, all the frames have an acceptable level of confidence against the instability; however, for the area with PGA = 0.45g, the 9 and 12 story frames should re-designed.

7 Summary

The probabilities of the dynamic instability obtained from the fragility curves. It is found that increasing the structural height, increases the probability of limit state exceedance, and reduces the confidence level for CP and GI. This is especially important for the high-rise frames which are more vulnerable to earthquakes with higher return period. This indicates that the existing building codes are not competent to design high-rise structures, and there is a quick need for an update in these codes. Methods such as performance-based design might be a solution. Also, the comparison of the probability of instability and collapse confidence level for different frames under different earthquake events shows that a higher relative hazard threshold is required for the high-rise buildings compared to the existing one.

Accounting for uncertainties in both structural modeling and record-to-record variability, probability of global instability lie in the range 111.5% for earthquake ground motions with a 2%/50 yrs. Similarly, the same scenario produces a probability of 420% in collapse prevention performance level. Compared to FEMA 223 [75] the computed probabilities seems to be high for the considered buildings. On the other hand, the target threshold for confidence level is normally set to be 90% for the LSs located in regions with a return period of 2,475 years. Therefore, the buildings with PGA = 0.35g have an acceptable level of
confidence against instability; however, those 9 and 12 story buildings with PGA = 0.45g should be re-designed.

**References**


A Author's Biography

**Seyed Abbas Hosseini** received his BSc in Civil Engineering, MSc in Earthquake Engineering, and currently is PhD candidate in Sharif University of Technology. He is also Assistant Professor at Islamic Azad University, Germi Branch, Iran. His research interests are finite element modeling of building frames, dynamic analysis, and seismic performance assessment of structures.

**Mohsen Ghaemian** is Professor in Civil Engineering Department of Sharif University of Technology. His current activity is on concrete technology, dynamic responses of gravity and arch dams. Dam reservoir interaction effects, seismic response of dams due to non-uniform excitations, and nonlinear behavior of concrete dams have been in his research interest in these recent years.

**Mohammad Amin Hariri-Ardebili**, PhD, is a Research Associate and adjunct faculty at the Department of Structural Engineering and Mechanics, University of Colorado, Boulder (USA). His main research interests are performance-based assessment of major structures (dams, towers and nuclear containment vessels), coupled systems mechanics, mathematical models, and machine learning in structural engineering. He is currently an Associate Editor for Frontiers in Built Environment, and Editor for Mathematical Problems in Engineering.
List of Figures

1  Seismicity of Iran; original data from Iranian Seismological Center for the period of 2006-2015, adopted from [38] ................................................................. 25
2  Plan view of the designed buildings; for 3, 6 and 9 story frames d1 = 4.0 m and d2 = 5.0 m; for 12 and 15 story frames d1 = 5.0 m and d2 = 6.0 m ................. 26
3  Relative interstory drift and the allowable values according to Iranian seismic design regulation ................................................................. 27
4  The model used for plastic hinge ................................................................. 28
5  Base shear vs. roof drift ratio; SP = spectral pattern, UP = uniform pattern, TP = triangular pattern ................................................................. 29
6  Acceleration response spectra for the selected ground motions .............. 30
7  IDA curves using parameters proposed in [42] .............................................. 31
8  Fragility curves based on two stiffness model assumptions ..................... 32
9  Impact of structural height on fragility curves ........................................... 33
10  Impact of dispersion on fragility curves ................................................... 34
11  The hazard map of the Tehran region; adopted from Zafarani et al [68] ........ 35
12  Probability of exceeding the LSs in an earthquake event with 2,475 years return period ................................................................. 36
Figure 1: Seismicity of Iran: original data from Iranian Seismological Center for the period of 2006-2015, adopted from [38]
Figure 2: Plan view of the designed buildings; for 3, 6 and 9 story frames $d_1 = 4.0 \text{ m}$ and $d_2 = 5.0 \text{ m}$; for 12 and 15 story frames $d_1 = 5.0 \text{ m}$ and $d_2 = 6.0 \text{ m}$
Figure 3: Relative interstory drift and the allowable values according to Iranian seismic design regulation
Figure 4: The model used for plastic hinge
Figure 5: Base shear vs. roof drift ratio; SP = spectral pattern, UP = uniform pattern, TP = triangular pattern
Figure 6: Acceleration response spectra for the selected ground motions
Figure 7: IDA curves using parameters proposed in [42]
Figure 8: Fragility curves based on two stiffness model assumptions
Figure 9: Impact of structural height on fragility curves
Figure 10: Impact of dispersion on fragility curves
Figure 11: The hazard map of the Tehran region; adopted from Zafarani et al. [68]
Figure 12: Probability of exceeding the LSs in an earthquake event with 2,475 years return period
List of Tables

1. Effective stiffness of selective beam and column sections in a nine story frame .............................................. 38
2. Instability roof drift for the frames based on [32] .................................................. 39
3. Detailed ground motion characteristics for IDA ................................................ 40
4. CP confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.35 g ......................................................... 41
5. GI confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.35 g ......................................................... 42
6. CP confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.45 g ......................................................... 43
7. GI confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.45 g ......................................................... 44
Table 1: Effective stiffness of selective beam and column sections in a nine story frame

<table>
<thead>
<tr>
<th>Element Properties</th>
<th>A</th>
<th>Has</th>
<th>SCE</th>
<th>elton</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sect</td>
<td>Reinforcement</td>
<td>P/(A&lt;sub&gt;f&lt;/sub&gt;)</td>
<td>K&lt;sub&gt;e&lt;/sub&gt;/K&lt;sub&gt;g&lt;/sub&gt;</td>
<td>K&lt;sub&gt;e&lt;/sub&gt;/</td>
</tr>
<tr>
<td>ion</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B 40X50</td>
<td>Top</td>
<td>4φ22+4φ18</td>
<td>0</td>
<td>0.3</td>
</tr>
<tr>
<td>Top</td>
<td>4φ18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bot</td>
<td>4φ18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B 40X45</td>
<td>Top</td>
<td>4φ20+4φ18</td>
<td>0</td>
<td>0.3</td>
</tr>
<tr>
<td>Top</td>
<td>4φ18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bot</td>
<td>4φ18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C 50X50</td>
<td></td>
<td>16φ25</td>
<td>0.198</td>
<td>0.398</td>
</tr>
<tr>
<td>C 45X45</td>
<td></td>
<td>12φ25</td>
<td>0.163</td>
<td>0.363</td>
</tr>
<tr>
<td>C 40X40</td>
<td></td>
<td>12φ25</td>
<td>0.138</td>
<td>0.338</td>
</tr>
</tbody>
</table>
Table 2: Instability roof drift for the frames based on [42]

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>Load pattern</th>
<th>Instability drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Triangular</td>
<td>0.073</td>
</tr>
<tr>
<td>6</td>
<td>Spectral</td>
<td>0.062</td>
</tr>
<tr>
<td>9</td>
<td>Spectral</td>
<td>0.059</td>
</tr>
<tr>
<td>12</td>
<td>Spectral</td>
<td>0.052</td>
</tr>
<tr>
<td>15</td>
<td>Spectral</td>
<td>0.049</td>
</tr>
</tbody>
</table>
Table 3: Detailed ground motion characteristics for IDA

<table>
<thead>
<tr>
<th>ID</th>
<th>$M_w$</th>
<th>Event</th>
<th>Fault</th>
<th>$R_{rup}$</th>
<th>Component</th>
<th>PGA [g]</th>
<th>PGV [cm/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.5</td>
<td>Imperial Valley</td>
<td>Strike-slip</td>
<td>12.6</td>
<td>H-E11140</td>
<td>0.364</td>
<td>34.5</td>
</tr>
<tr>
<td>2</td>
<td>6.5</td>
<td>Imperial Valley</td>
<td>Strike-slip</td>
<td>12.6</td>
<td>H-E11230</td>
<td>0.38</td>
<td>42.1</td>
</tr>
<tr>
<td>3</td>
<td>7.5</td>
<td>Kocaeli, Turkey</td>
<td>Strike-slip</td>
<td>17</td>
<td>ARC090</td>
<td>0.149</td>
<td>39.5</td>
</tr>
<tr>
<td>4</td>
<td>7.3</td>
<td>Landers</td>
<td>Strike-slip</td>
<td>21.2</td>
<td>CLW-TR</td>
<td>0.417</td>
<td>42.3</td>
</tr>
<tr>
<td>5</td>
<td>6.9</td>
<td>Loma Prieta</td>
<td>Strike-slip</td>
<td>14.5</td>
<td>CAP090</td>
<td>0.443</td>
<td>29.3</td>
</tr>
<tr>
<td>6</td>
<td>6.9</td>
<td>Kobe, Japan</td>
<td>Strike-slip</td>
<td>0.6</td>
<td>KJM000</td>
<td>0.821</td>
<td>81.3</td>
</tr>
<tr>
<td>7</td>
<td>6.9</td>
<td>Loma Prieta</td>
<td>Strike-slip</td>
<td>25.8</td>
<td>HDA255</td>
<td>0.279</td>
<td>35.6</td>
</tr>
<tr>
<td>8</td>
<td>7</td>
<td>Cape Mendocino</td>
<td>Thrust</td>
<td>18.5</td>
<td>RIO360</td>
<td>0.549</td>
<td>42.1</td>
</tr>
<tr>
<td>9</td>
<td>6.9</td>
<td>Loma Prieta</td>
<td>Strike-slip</td>
<td>25.8</td>
<td>HDA165</td>
<td>0.269</td>
<td>43.9</td>
</tr>
<tr>
<td>10</td>
<td>6.9</td>
<td>Loma Prieta</td>
<td>Strike-slip</td>
<td>28.2</td>
<td>HCH090</td>
<td>0.247</td>
<td>38.5</td>
</tr>
<tr>
<td>11</td>
<td>6.9</td>
<td>Kobe, Japan</td>
<td>Strike-slip</td>
<td>26.4</td>
<td>KAK090</td>
<td>0.345</td>
<td>27.6</td>
</tr>
<tr>
<td>12</td>
<td>6.5</td>
<td>Imperial Valley</td>
<td>Strike-slip</td>
<td>10.6</td>
<td>H-CXO225</td>
<td>0.275</td>
<td>21.2</td>
</tr>
<tr>
<td>13</td>
<td>6.5</td>
<td>Friuli, Italy</td>
<td>Thrust</td>
<td>-</td>
<td>A-TMZ270</td>
<td>0.315</td>
<td>30.8</td>
</tr>
<tr>
<td>14</td>
<td>6.7</td>
<td>Northridge</td>
<td>Blind thrust</td>
<td>20.8</td>
<td>MU2035</td>
<td>0.617</td>
<td>40.8</td>
</tr>
<tr>
<td>15</td>
<td>7.1</td>
<td>Duzce Turkey</td>
<td>Strike-slip</td>
<td>17.6</td>
<td>BOL000</td>
<td>0.728</td>
<td>56.4</td>
</tr>
<tr>
<td>16</td>
<td>7.3</td>
<td>Landers</td>
<td>Strike-slip</td>
<td>24.9</td>
<td>YER360</td>
<td>0.152</td>
<td>29.7</td>
</tr>
<tr>
<td>17</td>
<td>6.7</td>
<td>Northridge</td>
<td>Blind thrust</td>
<td>19.6</td>
<td>MUL009</td>
<td>0.416</td>
<td>59</td>
</tr>
<tr>
<td>18</td>
<td>7.5</td>
<td>Kocaeli, Turkey</td>
<td>Strike-slip</td>
<td>12.7</td>
<td>DZC270</td>
<td>0.358</td>
<td>46.4</td>
</tr>
<tr>
<td>19</td>
<td>6.5</td>
<td>Imperial Valley</td>
<td>Strike-slip</td>
<td>11.1</td>
<td>H-SHP270</td>
<td>0.506</td>
<td>30.9</td>
</tr>
<tr>
<td>20</td>
<td>6.5</td>
<td>Friuli, Italy</td>
<td>Thrust</td>
<td>-</td>
<td>A-TMZ000</td>
<td>0.351</td>
<td>22</td>
</tr>
</tbody>
</table>
Table 4: CP confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.35 g

<table>
<thead>
<tr>
<th>Stories</th>
<th>Modeling</th>
<th>k</th>
<th>C</th>
<th>ϕ</th>
<th>D</th>
<th>γ</th>
<th>γa</th>
<th>βUT</th>
<th>λ</th>
<th>Kx</th>
<th>C.L</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.100</td>
<td>0.855</td>
<td>0.023</td>
<td>1.091</td>
<td>1.036</td>
<td>0.3</td>
<td>0.299</td>
<td>4.495</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.100</td>
<td>0.85</td>
<td>0.025</td>
<td>1.100</td>
<td>1.036</td>
<td>0.3</td>
<td>0.327</td>
<td>4.192</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.100</td>
<td>0.84</td>
<td>0.02</td>
<td>1.195</td>
<td>1.049</td>
<td>0.35</td>
<td>0.283</td>
<td>4.151</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.100</td>
<td>0.83</td>
<td>0.02</td>
<td>1.198</td>
<td>1.049</td>
<td>0.35</td>
<td>0.326</td>
<td>3.753</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.100</td>
<td>0.71</td>
<td>0.03</td>
<td>1.238</td>
<td>1.065</td>
<td>0.4</td>
<td>0.628</td>
<td>1.790</td>
<td>0.963</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.100</td>
<td>0.76</td>
<td>0.03</td>
<td>1.304</td>
<td>1.065</td>
<td>0.45</td>
<td>0.574</td>
<td>2.014</td>
<td>0.978</td>
</tr>
<tr>
<td>12</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.100</td>
<td>0.66</td>
<td>0.03</td>
<td>1.187</td>
<td>1.074</td>
<td>0.43</td>
<td>0.554</td>
<td>2.054</td>
<td>0.980</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.100</td>
<td>0.73</td>
<td>0.03</td>
<td>1.201</td>
<td>1.074</td>
<td>0.43</td>
<td>0.601</td>
<td>1.863</td>
<td>0.969</td>
</tr>
<tr>
<td>15</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.100</td>
<td>0.63</td>
<td>0.03</td>
<td>1.227</td>
<td>1.084</td>
<td>0.45</td>
<td>0.561</td>
<td>1.983</td>
<td>0.976</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.100</td>
<td>0.66</td>
<td>0.03</td>
<td>1.230</td>
<td>1.084</td>
<td>0.45</td>
<td>0.643</td>
<td>1.685</td>
<td>0.955</td>
</tr>
</tbody>
</table>
Table 5: GI confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.35 g

<table>
<thead>
<tr>
<th>Stories</th>
<th>Modeling</th>
<th>k</th>
<th>C</th>
<th>φ</th>
<th>D</th>
<th>γ</th>
<th>γₐ</th>
<th>βᵤₜ</th>
<th>λ</th>
<th>Kₓ</th>
<th>C.L</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Haselton</td>
<td>3.136</td>
<td>0.123</td>
<td>0.823</td>
<td>0.023</td>
<td>1.091</td>
<td>1.036</td>
<td>0.3</td>
<td>0.252</td>
<td>5.066</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.116</td>
<td>0.82</td>
<td>0.025</td>
<td>1.100</td>
<td>1.036</td>
<td>0.3</td>
<td>0.276</td>
<td>4.765</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.127</td>
<td>0.75</td>
<td>0.02</td>
<td>1.195</td>
<td>1.049</td>
<td>0.35</td>
<td>0.251</td>
<td>4.495</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.114</td>
<td>0.78</td>
<td>0.02</td>
<td>1.198</td>
<td>1.049</td>
<td>0.35</td>
<td>0.306</td>
<td>3.929</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.106</td>
<td>0.71</td>
<td>0.03</td>
<td>1.238</td>
<td>1.065</td>
<td>0.4</td>
<td>0.585</td>
<td>1.966</td>
<td>0.976</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.111</td>
<td>0.74</td>
<td>0.03</td>
<td>1.304</td>
<td>1.065</td>
<td>0.45</td>
<td>0.543</td>
<td>2.058</td>
<td>0.980</td>
</tr>
<tr>
<td>12</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.091</td>
<td>0.75</td>
<td>0.03</td>
<td>1.187</td>
<td>1.074</td>
<td>0.43</td>
<td>0.540</td>
<td>2.113</td>
<td>0.983</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.082</td>
<td>0.74</td>
<td>0.03</td>
<td>1.201</td>
<td>1.074</td>
<td>0.43</td>
<td>0.723</td>
<td>1.429</td>
<td>0.924</td>
</tr>
<tr>
<td>15</td>
<td>Haselton</td>
<td>3.136</td>
<td>0.100</td>
<td>0.71</td>
<td>0.03</td>
<td>1.227</td>
<td>1.084</td>
<td>0.45</td>
<td>0.498</td>
<td>2.249</td>
<td>0.988</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>3.136</td>
<td>0.110</td>
<td>0.71</td>
<td>0.03</td>
<td>1.230</td>
<td>1.084</td>
<td>0.45</td>
<td>0.543</td>
<td>2.058</td>
<td>0.980</td>
</tr>
</tbody>
</table>
Table 6: CP confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.45 g

<table>
<thead>
<tr>
<th>Stories</th>
<th>Modeling</th>
<th>k</th>
<th>C</th>
<th>φ</th>
<th>D</th>
<th>γ</th>
<th>γ_α</th>
<th>β_UT</th>
<th>λ</th>
<th>K_x</th>
<th>C.L</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.100</td>
<td>0.91</td>
<td>0.041</td>
<td>1.781</td>
<td>1.022</td>
<td>0.3</td>
<td>0.811</td>
<td>0.986</td>
<td>0.839</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.100</td>
<td>0.91</td>
<td>0.041</td>
<td>1.472</td>
<td>1.022</td>
<td>0.3</td>
<td>0.677</td>
<td>1.590</td>
<td>0.944</td>
</tr>
<tr>
<td>6</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.100</td>
<td>0.90</td>
<td>0.040</td>
<td>1.088</td>
<td>1.030</td>
<td>0.35</td>
<td>0.502</td>
<td>2.308</td>
<td>0.990</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.100</td>
<td>0.89</td>
<td>0.041</td>
<td>1.299</td>
<td>1.030</td>
<td>0.35</td>
<td>0.617</td>
<td>1.718</td>
<td>0.957</td>
</tr>
<tr>
<td>9</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.100</td>
<td>0.81</td>
<td>0.052</td>
<td>1.491</td>
<td>1.039</td>
<td>0.4</td>
<td>1.007</td>
<td>0.367</td>
<td>0.644</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.100</td>
<td>0.84</td>
<td>0.051</td>
<td>1.154</td>
<td>1.039</td>
<td>0.4</td>
<td>0.726</td>
<td>1.185</td>
<td>0.881</td>
</tr>
<tr>
<td>12</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.100</td>
<td>0.78</td>
<td>0.045</td>
<td>1.090</td>
<td>1.045</td>
<td>0.43</td>
<td>0.658</td>
<td>1.392</td>
<td>0.918</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.100</td>
<td>0.82</td>
<td>0.051</td>
<td>1.129</td>
<td>1.045</td>
<td>0.43</td>
<td>0.727</td>
<td>1.158</td>
<td>0.877</td>
</tr>
<tr>
<td>15</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.100</td>
<td>0.75</td>
<td>0.039</td>
<td>1.093</td>
<td>1.051</td>
<td>0.45</td>
<td>0.600</td>
<td>1.563</td>
<td>0.941</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.100</td>
<td>0.77</td>
<td>0.043</td>
<td>1.096</td>
<td>1.051</td>
<td>0.45</td>
<td>0.647</td>
<td>1.396</td>
<td>0.919</td>
</tr>
</tbody>
</table>
Table 7: GI confidence level of frames under an earthquake event with 2,475 years return period and PGA = 0.45 g

<table>
<thead>
<tr>
<th>Stories</th>
<th>Modeling</th>
<th>k</th>
<th>C</th>
<th>Φ</th>
<th>D</th>
<th>γ</th>
<th>γₜ</th>
<th>βᵤₜ</th>
<th>λ</th>
<th>Kₛ</th>
<th>C.L</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.123</td>
<td>0.89</td>
<td>0.041</td>
<td>1.781</td>
<td>1.022</td>
<td>0.3</td>
<td>0.674</td>
<td>1.606</td>
<td>0.946</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.116</td>
<td>0.89</td>
<td>0.041</td>
<td>1.472</td>
<td>1.022</td>
<td>0.3</td>
<td>0.596</td>
<td>2.014</td>
<td>0.978</td>
</tr>
<tr>
<td>6</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.127</td>
<td>0.83</td>
<td>0.040</td>
<td>1.088</td>
<td>1.030</td>
<td>0.35</td>
<td>0.425</td>
<td>2.784</td>
<td>0.997</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.114</td>
<td>0.86</td>
<td>0.041</td>
<td>1.299</td>
<td>1.030</td>
<td>0.35</td>
<td>0.565</td>
<td>1.968</td>
<td>0.976</td>
</tr>
<tr>
<td>9</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.106</td>
<td>0.81</td>
<td>0.052</td>
<td>1.491</td>
<td>1.039</td>
<td>0.4</td>
<td>0.943</td>
<td>0.533</td>
<td>0.702</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.111</td>
<td>0.83</td>
<td>0.051</td>
<td>1.154</td>
<td>1.039</td>
<td>0.4</td>
<td>0.664</td>
<td>1.409</td>
<td>0.921</td>
</tr>
<tr>
<td>12</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.091</td>
<td>0.84</td>
<td>0.045</td>
<td>1.090</td>
<td>1.045</td>
<td>0.43</td>
<td>0.672</td>
<td>1.342</td>
<td>0.910</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.082</td>
<td>0.83</td>
<td>0.051</td>
<td>1.129</td>
<td>1.045</td>
<td>0.43</td>
<td>0.878</td>
<td>0.716</td>
<td>0.764</td>
</tr>
<tr>
<td>15</td>
<td>Haselton</td>
<td>1.927</td>
<td>0.100</td>
<td>0.81</td>
<td>0.039</td>
<td>1.093</td>
<td>1.051</td>
<td>0.45</td>
<td>0.557</td>
<td>1.726</td>
<td>0.948</td>
</tr>
<tr>
<td></td>
<td>ASCE</td>
<td>1.927</td>
<td>0.110</td>
<td>0.81</td>
<td>0.043</td>
<td>1.096</td>
<td>1.051</td>
<td>0.45</td>
<td>0.563</td>
<td>1.702</td>
<td>0.955</td>
</tr>
</tbody>
</table>