EFFECT OF STRUCTURAL MODELING UNCERTAINTIES ON SEISMIC PERFORMANCE OF STEEL MOMENT RESISTING FRAMES

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ABSTRACT

Prediction of structure’s response to seismic loads is a complex problem with many parameters involved that some of them can behave highly uncertain. Nonetheless it is needed to have a clear understanding of how these uncertainties affect structural seismic performance. In this matter, it is convenient to separate uncertainties into two categories: *aleatory* (due to variability of strong ground motions) and *epistemic* (related to structure's numerical model).

This Paper aims to investigate effect of structural uncertainties on seismic performance of steel moment resisting frames through extended IDA of a sample 5-storey frame. In this regards, uncertainties in damping, mass, yield strength and ultimate strength of structural steel have been considered as probabilistic variables. Latin Hypercube Sampling (LHS) has been used to create random realizations of structures. With the aid of reliability methods, different sources of uncertainty and their ranges of influence on seismic performance have been disaggregated

Considering results, it can be seen that uncertainties in selected parameters have important effect on seismic performance. Capacity and demand estimations based only on deterministic procedures may ignore some substantial points. Also including these uncertainties in performance calculations can considerably change probability of achieving desired performance at some levels.

INTRODUCTION

Incremental dynamic analysis (IDA), introduced by Vamvatsikos and Cornell (2002), is nowadays a widely used tool to study seismic performance of structures and has been discussed in many researches and technical reports (FEMA-350, 2000; FEMA-440, 2005; Vamvatsikos and Fragiadakis, 2010). Using this method is usually based on a deterministic numerical model of structure, which is affected only by aleatory uncertainties (known as record to record effect). But in a more developed method, which is named *extended IDA*; it is possible to perform IDAs with a probabilistic description of structural model. In such case, results will contain both aleatory and epistemic uncertainties.

Extended IDA has been subject for many researches in recent decade. Dolsek (2009) studied effects of epistemic uncertainties on seismic capacity of a 4-storey concrete moment resisting frame through extended IDA, with selecting a set of various structural modeling parameters as probabilistic variables. Zareian and Krawinkler (2007) suggested a probabilistic-based methodology for quantifying the collapse potential of structural systems, based on different sources of uncertainty and for desired levels of confidence.

Lignos et al. (2008) evaluated reliability of a 4-storey steel moment resisting frame against collapse caused by seismic loads in which they modeled moment-rotation characteristics of plastic hinges as probabilistic variables. Vamvatsikos and Fragiadakis (2010) also employed parameterized moment-rotation relationships with non-deterministic quadrilinear backbones for the beam plastic-hinges. They studied effect of these uncertainties on seismic performance of a 9-storey steel moment resisting frame using various statistical tools.

Although in case of steel moment resisting frames there have been diverse studies done to quantitate the effect of uncertainty in moment-rotation characteristics of members, evaluation of other structural modelling parameters has not been of particular interest yet. So results of new studies in this area can create a new viewpoint about effects of epistemic uncertainties on seismic performance. Also with proper assumptions made, one can consider performance aspects of structure in a more meaningful way and compare it to what is expected by governing codes and standards.

METHODOLOGY USED IN THIS PAPER

To be able to separate between epistemic and aleatory sources of uncertainty, it is needed to analyse structure in its deterministic form as well as probabilistic form. So on first level, structure is analysed through IDA for a selected set of earthquake records with its parameters set to central values (which will be named as *Base Structure* hereafter). Results of this level includes solely uncertainty due to record to record effect.

On the next level, assuming damping, efficient seismic mass, yield strength and ultimate strength of steel as probabilistic variables, a sufficient number of different realizations of structural model are generated. Then every single realization of structure is subjected to IDA for selected records. Results of this level includes effects of both aleatory and epistemic uncertainties.

In order to optimize the procedure of generating random realizations of structure, here Latin hypercube sampling (LHS) method (McKay et al., 1979) has been used. This technique uses a constrained sampling scheme instead of random sampling utilized by direct Monte Carlo method, and consequently will need significantly fewer simulations to cover desired probability space (Dolsek, 2009; Rajeev and Tesfamariam, 2012).

With putting results of these two levels together, and utilizing reliability methods, it will be feasible to separate between different sources of uncertainty and to define the extent that each one has affected seismic performance.

According to definitions existing in FEMA-350 (2000), performance limit states can be defined for steel moment resisting frames. Particularly, three limit states namely immediate occupancy (IO), collapse prevention (CP) and global instability (GI) are of prime interest.

After gathering pair values of intensity and demand measures for different performance levels on multiple record IDA curves, statistical characteristics of their distribution can be calculated. Central values (median or mean) will represent structural capacity on different performance levels and dispersion values (standard deviation) will represent the extent of uncertainty effects.

Also with assignment of a proper statistical distribution to pair values of demand and intensity, a cumulative distribution function will be available that helps to define the fragility function of structure for every desired performance level. In this matter, a convenient and widely-accepted assumption is using lognormal distribution (FEMA-P695, 2009; Lignos et al., 2008). With this procedure done, probability of exceedance for specific level of seismic demand can be calculated.

STRUCTURAL SYSTEM AND NUMERICAL MODELING

Structure under consideration is a 5-storey 3-bay steel special moment resisting frame. Height of stories is 3.20 m and width of bays is 5.0 m (Fig.1). Vertical and seismic loadings are applied according to Iranian National Code of Minimum Building Loads (2006) and Iranian Code of Practice for Seismic Resistant Design of Buildings (2005), respectively. Structural site is assumed to be located in a region of very high seismic risk (relative design base acceleration equal to 0.35g) and soil category corresponds to type II (which is very similar to category C of NEHRP classification). Also Iranian Steel Design Code (2008) is used to design the frame with implementation of ASD method. Based on modal analysis of frame, this structure has a fundamental period equal to 1.12 seconds.

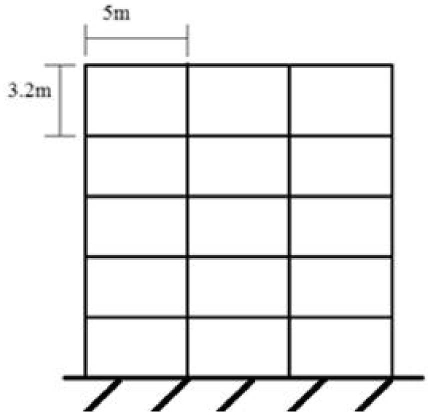

Figure 1. Geometrical properties of frame under consideration

OpenSees (2006) has been used to create mathematical model of moment frame and to perform analyses. Nonlinear beam-column elements have been used to constitute elements of frame and fiber section method is implemented to incorporate spread plasticity in nonlinear behaviour of elements. This method will form a member as a group of fibres each of which can have a uniaxial force-deformation behaviour defined by user. In this paper, Steel02 material from software library has been selected to describe the hysteretic behaviour of steel, and Fatigue material is used in order to define a limit value on deformations (i.e. for deformations exceeding this pre-defined value, fiber will fail).

Corotational method is used which performs an exact geometric transformation of beam stiffness and resisting force from the basic system to the global coordinate system (Mazzoni et al., 2007). Rayleigh command existing in OpenSees library is used to form classic damping matrice of structure. Also to form mass matrice, the assumption of concentrated nodal mass is used meaning that elements are weightless in their length but an amount of mass is assigned to each node considering adjacent lengths of loading, leading to formation of a diagonal mass matrice.

QUANTIFICATION OF UNCERTAINTIES

When using LHS method to generate a pseudo-random set of numbers for parameters, it is needed to have a specific statistical target distribution describing probability distribution function (PDF) of those parameters, as well as their correlation matrix.

To introduce PDF of ratio of equivalent viscous damping (ξ), here the work of Porter et al. (2002) is used. Porter et al. (2002) have compiled results of some researches done to estimate ξ for different kinds of structures and concluded that a reasonable value for coefficient of variation (C.O.V) for this parameter should be between 0.3 and 0.4. Based On that and some other studies (e.g. Mehanny and Ayoub, 2008) here it is chosen to assign a lognormal distribution with median of 0.05 and C.O.V of 0.4 for parameter ξ.

Ellingwood et al. (1980) with concluding the results of some other studies suggested that a proper way to describe dead loads as a probabilistic variable, is to assume it as a normal distribution with mean value equal to dead loads used in design procedure and a C.O.V equal to 0.1. The same suggestion has been used in this paper.

PDF of yield strength (F_y) and ultimate strength (F_u) of steel are taken as introduced in report No.177 of John A. Blume earthquake engineering center (Lignos and Krawinkler, 2009). As presented in this report, based on data gathered from results of tensile strength tests on flange coupons, statistical properties of PDFs for F_y and F_u are as shown in Table 1 (σ is standard deviation and ρ is correlation coefficient):

Table 1. Statistics of material yield strength from flange coupon tests (Lignos and Krawinkler, 2009)

Mean F_y (MPa)	σ_{Fy} (MPa)	Mean F_u (MPa)	σ_{Fu} (MPa)	$\rho_{Fu,Fy}$
310.3	35.80	455.7	29.60	0.851

Based on statistical target distributions described above and using LHS method, a set of 75 random realizations of structure are generated. For a better depiction of how values of probabilistic parameters vary for different realizations of structure, their individual PDFs are described in Table 2.

Table 2. Statistical characteristics of distributions used for uncertain parameters

Parameter	Min	Median	Max
Ratio of equivalent viscous damping (%)	1.69	4.63	14.1
Dead load (KN)	19.4	25.0	32.1
Yield strength (Mpa)	235	309	401
Ultimate strength (Mpa)	437	501	585

A set of 12 records have been selected which correspond to soil category C (based on NEHRP classification) and are all free of near-field effects. General properties of these records are presented in Table 3. Also to see how they match with seismicity of assumed region for structure, their linear acceleration spectra are depicted against 475 years uniform hazard spectra (UHS) of the region in Fig.2 (UHS curve is extracted from seismic hazard analysis project established by President deputy strategic planning and control (2006)). As can be seen in Fig.2, mean spectral acceleration of selected records is well correspondent to UHS of the area. Hence it can be said that these records, in an average sense, are representative of seismicity for assumed area.

Table 3. Earthquake Records Used for Nonlinear Time History Analyses

Event	Year	Station	Moment Magnitude	Distance (km)
Cape Mendocino	1992	Eureka	7.01	42
Duzce	1999	Lamont 1062	7.14	10.2
Imperial Valley	1979	Superstition	6.53	25.23
Kern County	1952	Taft	7.36	38.89
Kobe	1995	Nishi-Akashi	6.9	8.12
Kocaeli	1999	Arcelik	7.51	13.52
Loma Prieta	1989	Gilroy	6.93	9.96
Loma Prieta	1989	Anderson Dam	6.93	20.26
Manjil	1990	Abbar	7.37	12.97
Northridge	1994	L.A Baldwin Hills	6.69	29.88
Northridge	1994	Obregon Park	6.69	37.36
Victoria	1980	Cerro Prieto	6.33	14.37

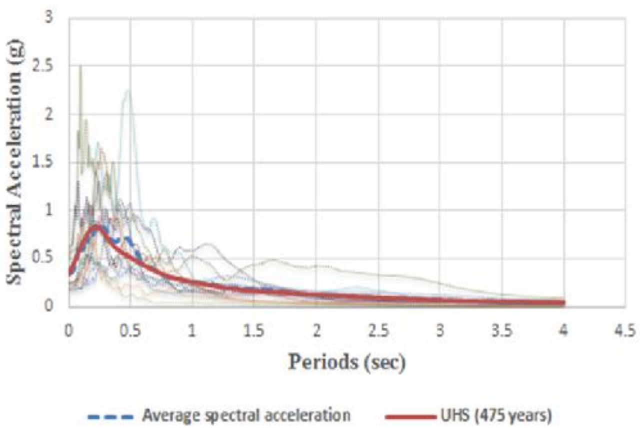

Figure 2. Spectral Acceleration of selected records, and their average against 475 years UHS of region

RESULTS OF PERFORMANCE EVALUATIONS

For the first part of results, summarized IDA curves (median, median plus and median minus one standard deviation) for both cases of including and excluding epistemic uncertainties (Base Case and Uncertain Case) are depicted in Fig.3. From this Figure, it can be seen that epistemic uncertainties have considerable effect on all curves with decrease in structural capacity, and this decrease has a direct relation to the level of seismic demand.

Another important point from Fig.3 is that the median of structural capacities in uncertain realizations is not equivalent to capacities of Base Structure. In other words, as has been discussed in details in Vamvatsikos and Fragiadakis (2010), convenient assumption that the median-parameter model will produce the median seismic performance is not necessarily true. In addition to curves, seismic capacities corresponding to different performance limit states are presented in Tables 4 and 5.

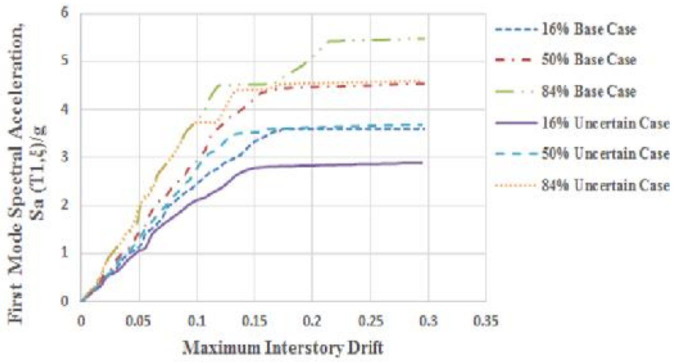

Figure 3. Summarized IDA curves for deterministic structural model (Base Case) and uncertain structural model (Uncertain Case)

Table 4. Summarized results of IDAs for different performance levels (Base Case)

Performance Level	16% (Sa(T1,ξ)/g)	50% (Sa(T1,ξ)/g)	84% (Sa(T1,ξ)/g)
IO	0.52	0.57	0.72
CP	2.47	2.94	3.80
GI	4.40	4.54	5.49

Table 5. Summarized results of IDAs for different performance levels (Uncertain Case)

Performance Level	16% (Sa(T1,ξ)/g)	50% (Sa(T1,ξ)/g)	84% (Sa(T1,ξ)/g)
IO	0.45	0.54	0.63
CP	2.19	2.79	3.51
GI	2.89	3.69	4.49

As mentioned previously, dispersion of capacity values given demand, represents the extent of uncertainty effects in that demand state. Results of Tables 4 & 5 can be used to calculate this dispersion, but to be able to separate between aleatory and epistemic uncertainties it will be necessary to use reliability methods. Here, simply the assumption of square-root-sum-of-squares (SRSS) is implemented. In other words, if β_U is dispersion due to epistemic uncertainty and β_R is dispersion due to aleatory uncertainty, then simultaneous effect of both of them will be calculated as (FEMA-350, 2000):

$$_{UR} = \sqrt{\ _U^2 + \ _R^2} \tag{1}$$

$$= \frac{1}{2}(\ln S_a^{84\%} - \ln S_a^{16\%}) \tag{2}$$

With these concepts, values of β parameters are estimated as shown in Table 6:

Table 6. Values of dispersion due to different uncertainties

Performance Level	β_U	β_R	β_{UR}
IO	0.17	0.16	0.05
CP	0.23	0.22	0.09
GI	0.22	0.11	0.19

Considering values of Table 6, it can be seen that dispersion due to epistemic uncertainties for IO and CP performance levels are restricted and almost negligible, but when it comes to GI limit state these dispersions become considerably more effective. Some other papers with subject of quantification of uncertainties in moment-rotation characteristics of frame members (e.g. Lignos et al., 2008), have reported these dispersions in order of 0.3. Hence comparing with results of this study, a reduction in extent of epistemic uncertainty can be seen.

As another part of results, fragility curves are produced that represent a probabilistic description for different performance levels of frame. Figs. 4 to 6 depict these fragility functions for both Base Case and Uncertain Case of structural model. Also in order to have a better sight into how structural capacities differ from one performance level to the other, fragility curves of three different limit states (IO, CP and GI) are shown together for base case in Fig.7.

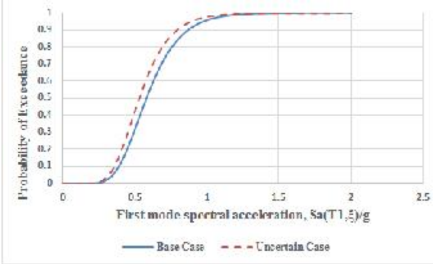

Figure 4. Fragility curves of Base and Uncertain cases for IO performance level

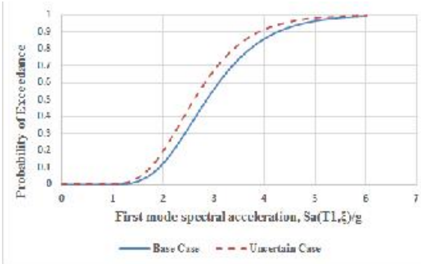

Figure 5. Fragility curves of Base and Uncertain cases for CP performance level

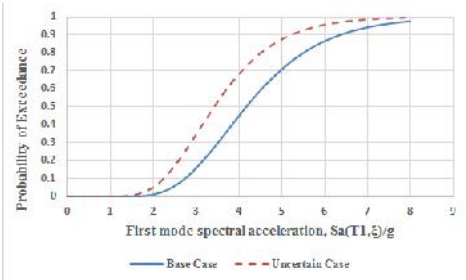

Figure 6. Fragility curves of Base and Uncertain cases for GI limit state

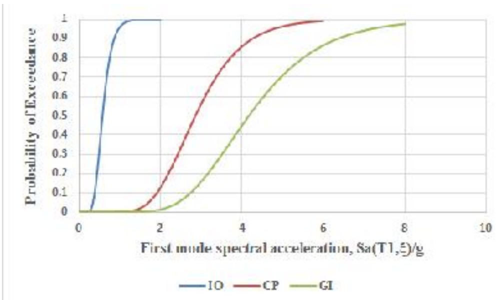

Figure 7. Fragility curves of different performance limit states in comparison

As it can be seen in Figs. 4 to 6, fragility curves of base and uncertain cases are different from each other, especially in GI limit state this difference becomes very effective. From another aspect, comparing fragility curves of different performance levels in Fig.7, a considerable distant can be seen between CP performance level (indicating severe damages) and GI limit state (indicating global instability). This means a desirable performance capacity against collapse exists for frame. For example, an earthquake which imposes a spectral acceleration equal to 2.87g on this structure has a 50% probability of exceedance for CP limit state, whereas it only has 13% probability of exceedance for GI limit state.

Following the methodology introduced by Zareian and Krawinkler (2007), some other results can be found which show important changes in probability of exceedance for different performance levels. Mentioned methodology gives the ability to calculate median of structural collapse capacity with incorporation of epistemic uncertainties for an arbitrary level of confidence.

If spectral acceleration gained from hazard analysis of structure's location shows the possible intensity of future earthquake for a specific hazard level, then with aid of fragility curve one can estimate probability of collapse for the given structure solely based on aleatory uncertainties. In the case of taking epistemic uncertainties into account, collapse capacity can be calculated for a desirable level of confidence. For example if confidence level of 90% is sought, collapse capacity will be calculated as following (Zareian and Krawinkler, 2007):

$$\ln(\eta^{90\%}) = \ln(\eta^{50\%}) - 1.28\beta_U \tag{3}$$

In Eq.3, η is median capacity, uppercase index shows desirable confidence level and β_U is dispersion due to structural modelling uncertainties. These needed quantities can be gained from extended IDA results. Of course using this methodology necessitates selected set of ground motion records to be representative of seismic properties for selected region. Here it is assumed that this assumption is true for 12 selected ground motion records.

Based on Iranian Code of Practice for Seismic Resistant Design of Buildings (2005), spectral acceleration suggested for this building will be approximately equal to 0.74g. If this value of spectral acceleration is taken on fragility curve shown in Fig.6 (Base Case), probability of collapse will be zero. In other words, when no structural uncertainty is assumed, probability of collapse is zero for possible future earthquake with return period of 475 years.

But considering that $\eta^{50\%}$=4.54g and β_U=0.19 for GI limit state (Tables 4 and 6), using Eq.3, $\eta^{90\%}$ will be calculated equal to 3.56g. If this value of spectral acceleration is taken on fragility curve of Fig.6, probability of collapse will be about 31 percent. Having the same procedure done for other performance levels, probability of exceedance for IO and CP with confidence level of 90% will be 39 and 50 percent respectively.

CONCLUSIONS

In this paper, effect of epistemic uncertainties on seismic performance of a 5-storey steel moment frame has been investigated. Selected probabilistic parameters for structural model are equivalent viscous damping ratio, seismic efficient mass, yield and ultimate strength of steel. LHS method is used to generate random realizations of structure, which are then analysed through Extended IDA. Reliability methods are implemented to separate between different sources of uncertainty.

Considering the results, a significant decrease in seismic capacity of structure can be seen due to incorporation of epistemic uncertainties. Dispersions caused by epistemic uncertainties are reduced when compared with a number of previous studies which aimed at studying moment-rotation characteristics of frame members. Also, an effective difference is observed between fragility curves of CP and GI limit states which represents a desirable performance capacity for special steel moment frame against seismic collapse. Moreover, using statistical methodology, it is observed that with conception of confidence level and including structural modelling uncertainties, probability of exceedance for a particular performance level can significantly change.

In Overall, results of this study indicate that incorporation of structural modelling uncertainties can make some substantial changes in safety considerations and damage control plans of a building against a future earthquake event.

REFERENCES

Dolsek M (2009) Incremental Dynamic Analysis with Consideration of Modelling Uncertainties, *Earthquake Engineering and Structural Dynamics*, 38(6):805–825

Ellingwood B, Galambos TV, MacGregor JG and Cornell CA (1980) Development of a Probability-Based Load Criterion for American National Standard A58, National Bureau of Standards, Washington, DC

FEMA-350 (2000) Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, Report No. 350, Federal Emergency Management Agency, Washington, DC

FEMA-440 (2005) Improvement of Nonlinear Static Seismic Analysis Procedures, Report No. 440, Federal Emergency Management Agency, Washington, DC

FEMA-P695 (2009) Quantification of Building Response Factors, Prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, DC

Iranian Code (2005) Practice for Seismic Resistance Design of Building, Standard No 2800-05, 3rd edition, Building and Housing Research Center, Tehran, Iran

Iranian National Code of Minimum Building Loads (2006) Standard No. 519-2000

Iranian Steel Design Code (2008) Standard No. 10 of National Codes for Structural Design

Lignos DG and Krawinkler H (2009) Sidesway Collapse of Deteriorating Structural Systems Under Seismic Excitations, Report No 177, John A Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University

Lignos D, Zareian F and Krawinkler H (2008) Reliability of a 4-Story Steel Moment-Resisting Frame against Collapse due to Seismic Excitations, *ASCE Structures Congress*, Long Beach, CA

Mazzoni S, McKenna F, Scott MH, Gregory L, Fenves GL and Jeremic B (2007) Open System for Earthquake Engineering Simulation (OpenSees): OpenSees Command Language Manual

McKay MD, Conover WJ and Beckman RA (1979) Comparison of Three Methods for Selecting Values of Input Variables in the Analysis of Output from a Computer Code, *Technometrics*, 21(2):239–245

Mehanny SSF, Ayoub AS (2008) Variability in Inelastic Displacement Demands: Uncertainty in System Parameters versus Randomness in Ground Records, *Engineering Structures*, 30:1002–1013

Open system for earthquake engineering (OpenSees) (2006) Berkley: Pacific Earthquake Engineering Research Center, University of California, Available at: http://opensees.berkeley.edu

Porter KA, Beck JL and Shaikhutdinov RV (2002) Sensitivity of Building Loss Estimates to Major Uncertain Variables, *Earthquake Spectra*, 18(4): 719–43

President Deputy Strategic Planning and Control, Bureau of Technical Execution System, Islamic Republic of Iran (2006), Seismic Hazard Analysis of I.R.IRAN Phase I-Greater Tehran Region, Available at: http://iranhazard.mporg.ir/

Rajeev P and Tesfamariam S (2012) Seismic Fragilities of non-Ductile Reinforced Concrete Frames with Consideration of Soil Structure Interaction, *Soil Dynamics and Earthquake Engineering*, 40:78–86

Vamvatsikos D and Fragiadakis M (2010) Incremental Dynamic Analysis for Estimating Seismic Performance Sensitivity and Uncertainty, *Earthquake Engineering and Structural Dynamics*, 39(2):141–63

Vamvatsikos D and Cornell CA (2002) Incremental Dynamic Analysis, *Earthquake Engineering and Structural Dynamics*, 31(3):491-514

Zareian F and Krawinkler H (2007) Assessment of Probability of Collapse and Design for Collapse Safety, *Earthquake Engineering and Structural Dynamics*, 36(13), 1901-1914