Fragility curves for mixed concrete/steel frames subjected to seismic excitation

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Fragility curves of complex concrete/steel frames subjected to seismic excitation

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Abstract

The reliable assessment of structural damage after an earthquake event is essential to organize the emergency response and to facilitate the structural and economic losses. Fragility curves are basic components in the process of earthquake loss estimation. They give the probability of exceeding a considerable number of damage states as a function of an intensity measures (IM) such as ground motion. In this paper fragility curves were developed for complex structures along their height. Those structures consist of two parts, a lower one made by reinforced concrete and the upper one made by structural steel. Fragility curves for concrete or for steel structures were proposed in literature, although the resulting fragility curves of a combination of those two structural systems is the main propose of this study. Parametric numerical results of moment resisting frames comprise of mixed in height concrete/steel material are presented and discussed.

Keywords: seismic fragility curves, complex concrete/steel frames, vulnerability curves, complex structural systems

1. Introduction

The methods to estimate seismic fragility functions can be grouped in three categories. The first is empirical, the second analytical, and last one expert opinion methods. However efforts which combine two of these approaches are also presented.

Empirical methods perform regression analysis of observed seismic performance with seismic excitation. The work by Wesson et al. [1] employs a large database of insurance claims from the 1994 Northridge earthquake, a rare occurrence in the public domain. The GEM Vulnerability Consortium, (GVC), [2], contributes to empirical methods since try to harmonize a variety of damage scales and create one that can be applied globally. US Geological Survey’s Prompt

Assessment of Global Earthquakes for Response (PAGER) project offers another approach to empirical vulnerability. In this project a statistical calculation determining probable past conditions of whole-earthquake fatality and economic losses, applying parametric vulnerability functions to estimate the number of people shaken at various levels of earthquakes intensity.

In analytical methods numerical calculations in order to evaluate damage or loss are used. A lot of work of analytical approaches was done, one description is presented in ATC-58 [3] which is the most recent and shows the tendency that the relative area of research is imprinted to the guidelines. In analytical approaches details such as the construction material, lateral force resisting system, height category, occupancy category, the area of building and all structural and non structural component are taken into consideration. Representative software that applies analytical approaches are HAZUS-MH [4]-[5], PACT [6], EQRM [7], ELER [8], CAPRA [9] and SELENA [10].

An analytical derivation of seismic vulnerability functions is presented in the the work by Porter et al. [11]-[13]. In the work of Silva et al [14] a study of static and dynamic procedures for estimating the nonlinear response of buildings has been carried out to evaluate the impact of the chosen methodology on the resulting capacity, fragility, vulnerability and risk outputs.

Hybrid methods combine statistical data with appropriately processed results from nonlinear dynamic or static analyses that permit interpolation of statistical data to PGAs and/or spectral displacements for which no data is available. This approach is implemented in the work of Kappos and Panagopoulos [15] in which they derived capacity curves and vulnerability (fragility) curves in terms of peak ground acceleration (PGA), as well as spectral displacement, for all types of R/C buildings that are common in Greece. Siqueira et al [16] developed fragility curves for isolated bridges in eastern Canada using experimental results.

Statistical procedures for contracting earthquake damage fragility functions are developing by Lallemant et al [17]. Noh et al [18] developed fragility functions derived from the wavelet-based damage sensitive feature *(DSF*). Gernay et al [19], proposed a fire fragility curves for steel buildings in a community context. Mitropoulou and Papadrakakis, [20], worked on developing fragility curves based on neural network IDA predictions.

A lot of studies have been conducted in developing fragility curves for concrete or steel structures. However fragility curves for material irregularity in height concrete-steel frame structures (complex concrete-steel frame structures) is very limited. In the work of Papageorgiou and Gantes, [21], dynamic response of elastic multi-degree of freedom structures that are irregular in height, consisting of two parts, a lower part made of concrete and an upper part, made of steel is presented. Skalomenos et al. [22] obtain fragility curves for three typical concrete filled steel tube, CFT, in plane moment-resisting frames, MRFs, designed according to European codes, for various levels of modeling sophistication through nonlinear time-history analyses. Güneyisi, [23] investigated the seismic reliability of three-storey and eight-storey steel moment resisting frames before and after retrofitting with buckling restrained braces (BRBs) in terms of seismic fragility and risk analysis. He developed fragility curves from the natural ground motions with low and high a/v ratio, (peak ground acceleration divided by the peak ground velocity). Maley et al., [24], applied the current code design recommendations for mixed MRF buildings. They designed a series of 8-storey mixed MRF systems of steel and reinforced concrete (RC) construction using both the direct displacement-based design (DDBD) method Priestley et al.,[25] and the ASCE-SEI 7-10, [26], equivalent lateral force (ELF) method. Sivapathasundaram and Mahendran, [27], developed fragility curves for localized pull-through failures of thin steel roof battens.

In this paper seismic fragility curves of complex concrete-steel frame structures is developed and compared with those corresponding to only concrete or only steel moment resisting frame structures. These structures consist of concrete frames over the lower storey and steel frames over the upper. Such buildings exist because a building might initially be constructed in RC and then some years later, in order to add new levels, additional storey in steel are added. Due to different time of construction those buildings are usually designed with different design approaches. However, if the lower part of building was constructed after the 90's then both parts are designed according to Eurocode, and can be considered from its origin, as a mixed building. Two types of mixed buildings were considered. The first type both parts of structure are designed with current codes, but in different time, and can be considered as a mixed building. The second type, the lower part of reinforced concrete is existed a lot of years before and low code requirements were used during its contraction while the upper steel part is designed according to current code.

2. Theoretical background and codes of developing fragility functions

The probability of structure being in or exceeding a given damage state, ds, is modeled as a cumulative lognormal distribution. For structural damage, given the spectral acceleration, a_d, the probability of being in or exceeding a damage state, ds, is modeled as:

$$P\left[ds|a_d\right]=\Phi\left[\frac{1}{\sigma_{a,ds}}\ln\left(\frac{a_d}{\overline{S}_{a,ds}}\right)\right] \tag{1}$$

where:

$\overline{S}_{a,ds}$ is the median value of spectral acceleration at which the building reaches the threshold of the damage state, ds,

$\sigma_{a,ds}$ is the standard deviation of the natural logarithm of spectral acceleration of damage state, ds,

Φ is the standard normal cumulative distribution function.

The median value of spectral acceleration, $\overline{S}_{a,ds}$, and the standard deviation of the natural logarithm of spectral acceleration, $\sigma_{a,ds}$, are calculated through a number of dynamic analyses for different earthquakes of building when reaches at a given damage state, d_s. Nevertheless, non linear static procedures can be applied in order to calculate those parameters. Furthermore, fragility curves can be represented with other intensity measures such as spectral displacement or the earthquake intensity.

In HAZUS MR4, Technical Manual the spectral displacement or acceleration is calculated using the classical push over method combine with capacity spectrum method. Non linear static procedures represent a simplified approach for the assessment of the seismic behavior of structures, included in guidelines such as the ATC-40, [28], FEMA-440, [29], and Eurocode 8, [30], in Europe.

In this study the median value of spectral acceleration, $\overline{S}_{a,ds}$, and the standard deviation of the natural logarithm of spectral acceleration, $\sigma_{a,ds}$, are calculated through a number of non linear dynamic analyses for different earthquakes. The procedure of calculating the fragility curves are as follows:

Step 1. Initially, the damage states of the building are determined. The inter-story drift ratio was chosen as a measure of damage states. Three damage states levels were determined. The first damage state level, $ds_{,1}$, is when the inter-story drift ratio is reached at 1%. The second and third damage state level, $ds_{,2}$, $ds_{,3}$, corresponds to the inter-story drift ratio of structure equal to 1.5% and 2% respectively. Moreover, earthquake excitations records that are compatible with the seismicity of the region in which the building is located are chosen. Specifically, in this study 10 erthquae excitations records were used.

Step 2. For the given damage stage $ds_{,i}$ with an iterative procedure apply non linear time history analysis for each earthquake excitation. Find the maximum earthquake acceleration that first inter-story drift ratio of structure reaches the limit of damage stage, ds,i. Repeat the above iterative procedure for the next damage state. Save all maximum accelerations in a group for each damaged state.

Step 3. For each damage state, $ds_{,i}$ calculate the median value of spectral acceleration, and the standard deviation of the natural logarithm of spectral acceleration. Calculate the probability of being in or exceeding a damage state, $ds_{,i}$, according to Eq. (1). Plot the fragility curve for each damage stage ds,i.

The general scheme which is followed to extract the fragility curves is shown in figure 1.

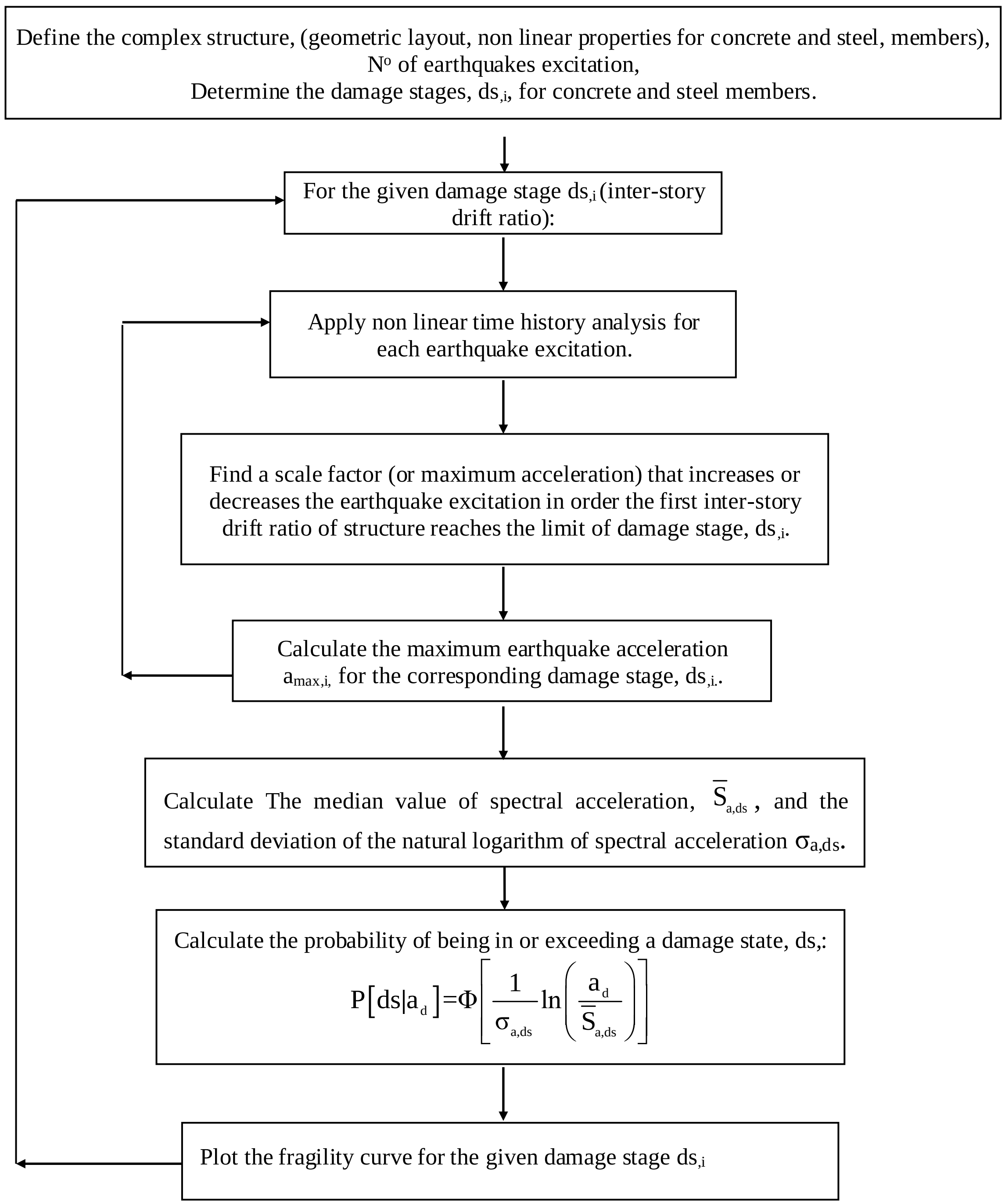

Figure 1 Chart diagram procedure of calculating the fragility curve.

Fragility curves of complex concrete/steel frame-Numerical applications

The complex concrete/steel frame consists of five story building; the first three are by concrete and the next two by steel. This is a typical application in existing concrete structure where the owners add one or two floors along with its high in elevation. Because of different time of construction the concrete part is supposed to designed by old Greek Code regulation while the upper part of steel is designed with Eurocode 3, EC3. Two models were examined. In the first one both parts of structure are designed with current codes, but in different time. In the second model the lower part of reinforced concrete was build a lot of years before with low code requirement and the upper steel part is designed according to current code.

The material distribution of complex concrete/steel frame is shown in figure 2.

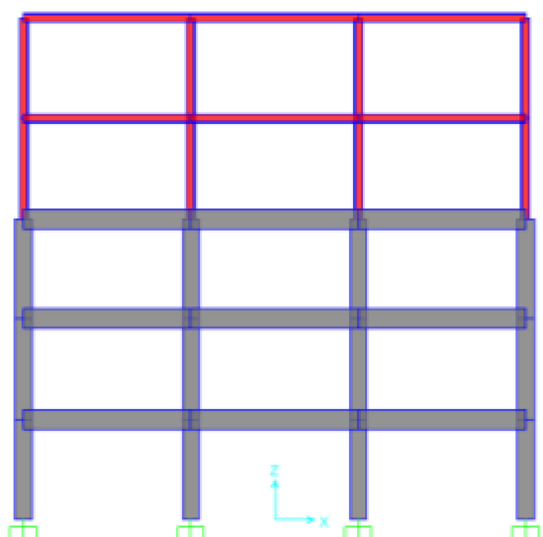

Figure 2 The complex concrete/steel frame and the material distribution. First three stories by concrete and the next two by steel.

The fixed plane frame has three openings of 5 m and the typical story high is 3m. All the vertical loads applied on the beams as distributed loads. The dead loads are 40 kN /m while the live load 15 kN /m.

The materials used for lower part are concrete, C20/25, with steel reinforcement B500C for the first model and C16/20, with steel reinforcement S400 for the second model. The upper part (fourth and fifth story) is structural steel of S275 grade for both models.

For the first model the concrete column sections are 50x50 cm with 8Φ18 longitudinal reinforcement per side and Φ10/10 vertical reinforcement. The beams section is 60x30 cm with

4Φ16 low and 2Φ12 at the top longitudinal reinforcement while has Φ10/15 vertical reinforcement. For the second model concrete column sections are 35x35 cm with 4Φ14 longitudinal reinforcement per side and Φ8/20 vertical reinforcement. The beams section is 60x30 cm with 3Φ14 low and 2Φ12 at the top longitudinal reinforcement while has Φ8/20 vertical reinforcement. The upper part for both models the steel columns are HEB260 and the steel beams HEA 260. The geometry layout and the members sections for first model are shown in figure 3.

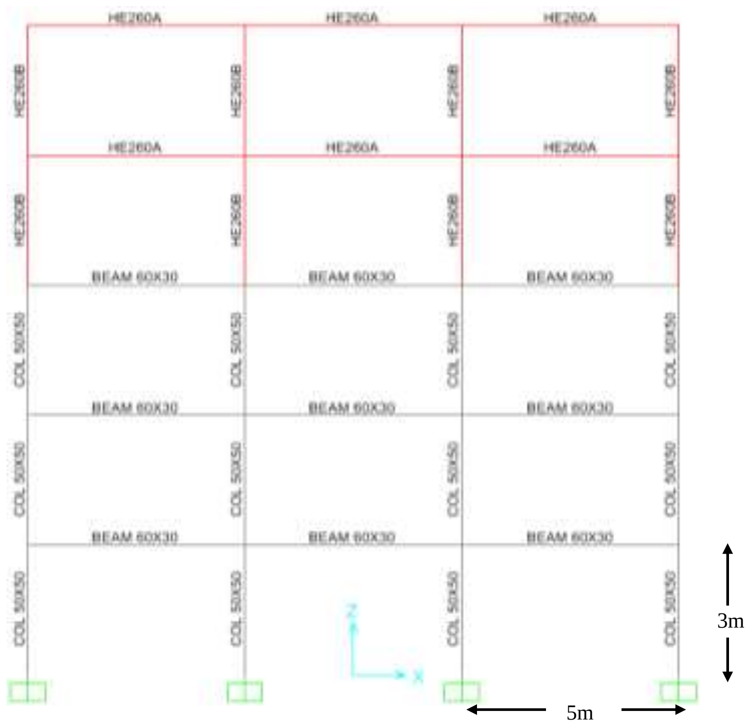

Figure 3 The geometry layout of complex concrete/steel frame and frame sections.

In order to calculate the fragility curve of the complex concrete/steel frame ten earthquake excitations were used. The record characteristics of the earthquakes are shown in table 1.
The inter-story drift ratio was chosen as a measure of damage states. Three damage states levels were determined. The first damage state level, $ds_{,1}$, is when the inter-story drift ratio is reach at 1%. The second and third damage state level, $ds_{,2}$, $ds_{,3}$, corresponds to the inter-story drift ratio of structure equal to 1.5% and 2% respectively.
For the non linear time history analysis a concentrated non linear behavior at the edge of members was considered. The moment-rotation relationship was calculated directed by the

software program SAP 2000, [31]. Takeda hysteretic model was used for the dynamic non linear behavior of elements. Hilber-Hughes-Taylor time integration procedure was chosen with parameters γ=0.5, β=0.25 and α=0.

Table 1. Recorded earthquake ground motions.

No.	Date	Record Name	Comp.	Station Name	PGA (g)
1	1980/06/09	Victoria, Mexico	N045	6604 Cerro Prieto	0.621
2	1992/04/25	Cape Mendocino	NS	89324 Rio Dell Overpass	0.549
3	1978/08/13	Santa Barbara	N048	283 Santa Barbara Courthouse	0.203
4	1978/08/13	Santa Barbara	N138	283 Santa Barbara Courthouse	0.102
5	1999/09/20	Chi-Chi, Taiwan	NS	TCU095	0.712
6	1994/01/17	Northridge	EW	90021 LA - N Westmoreland	0.401
7	1989/10/18	Loma Prieta	EW	58065 Saratoga - Aloha Ave	0.512
8	1992/06/28	Landers	NS	22170 Joshua Tree	0.284
9	1976/09/15	Friuli, Italy	EW	8014 Forgaria Cornino	0.26
10	1994/01/17	Northridge	NS	90019 San Gabriel - E. Gr. Ave.	0.256

Based on the procedure shown in figure 1 the fragility curves of complex concrete/steel frame for the three damage states were obtained. After performing the dynamic analysis of both models the structural fragility curve parameters, median and lognormal standard deviation, $\overline{S}_{a,ds}$, $\sigma_{a,ds}$ of spectral acceleration of three structural damage states d_s, were calculated and shown in table 2.

Table 2. Fragility curve parameters, median and lognormal standard deviation, $\overline{S}_{a,ds}$, $\sigma_{a,ds}$ of spectral acceleration of three structural damage states d_s

Damage state (interstory drift ratio)	**Median of spectral acceleration $\overline{S}_{a,ds}$**		**Lognormal standard deviation of spectral acceleration $\sigma_{a,ds}$**	
	1st Model, Current code	2nd Model, Low code	1st Model, Current code	2nd Model, Low code
ds,1 (1%)	0.486	0.243	0.473	0.419
ds,2 (1.5%)	0.751	0.364	0.529	0.432
ds1 (2%)	0.961	0.521	0.534	0.456

Using the values of fragility curve parameters shown in table 2 the fragility curves of complex concrete/steel frame for the two models for the three damage states were obtained and shown in figures 4 and 5.

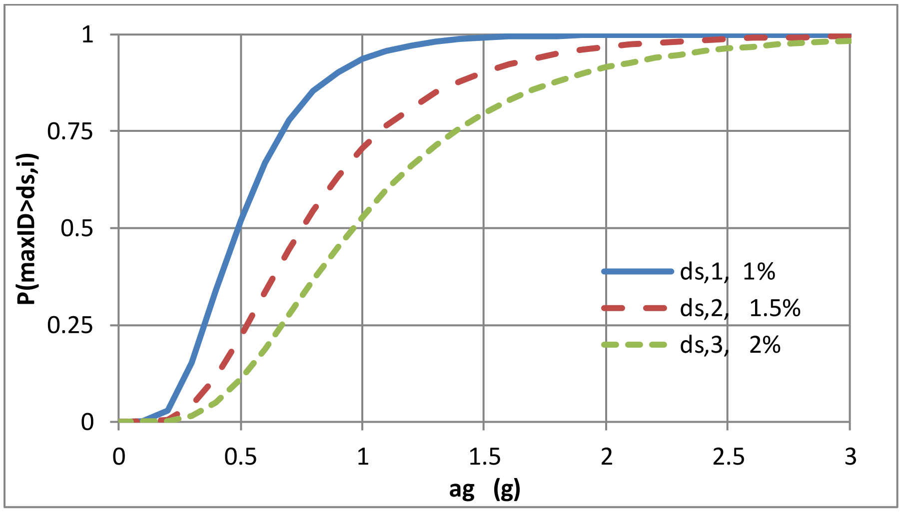

Figure 4 Fragility curves of complex concrete/steel frame (1st model with current code) for the three damage stages

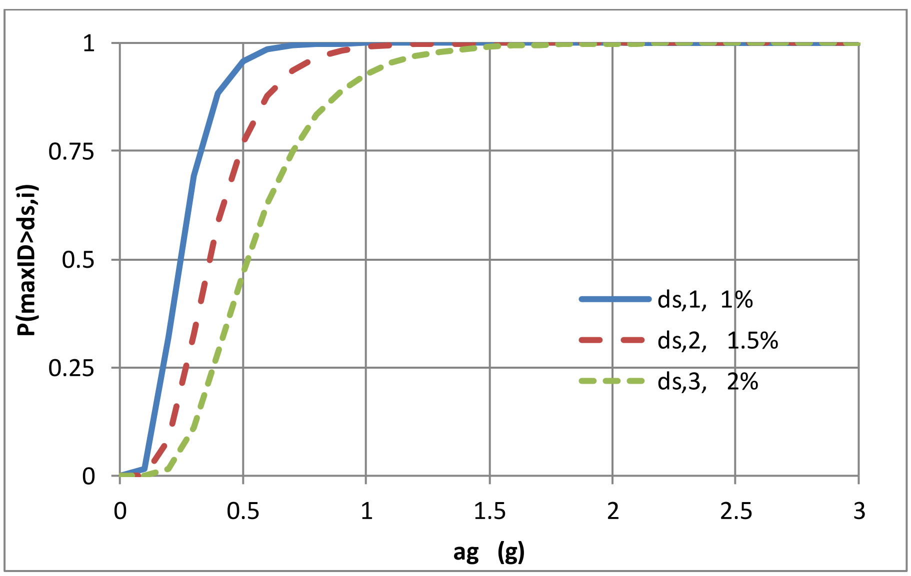

Figure 5 Fragility curves of complex concrete/steel frame (2nd model with low code) for the three damage stages

The hazus methodology proposes values for the fragility curve parameters, median and lognormal standard deviation, for only steel moment resistant frame and only reinforced concrete frame with medium height, 4-7 floors and designed with high-code seismic design level. Those parameters are shown in table 3.

Table 3. Fragility curve parameters, median and lognormal standard deviation, $\bar{S}_{a,ds}$, $\sigma_{a,ds}$ of spectral acceleration proposed in Hazus methodology.

Damage state (interstory drift ratio)		Median of spectral acceleration $\bar{S}_{a,ds}$	Lognormal standard deviation of spectral acceleration $\boldsymbol{\sigma_{a,ds}}$
S1M	ds,1, Moderate (0.8%)	0.26	0.64
	ds,2, Extensive (2%)	0.62	0.64
	ds,3, Complete (5.3%)	1.43	0.64
C1M	ds1, Moderate (0.67%)	0.27	0.64
	ds2, Extensive (2%)	0.73	0.64
	ds,3, Complete (5.3%)	1.61	0.64

Based on the values of fragility curve parameters proposed in Hazus methodology, shown in table 3, the fragility curves for steel and concrete structures for high-code seismic design level are drawn for each damage state. The fragility curves for concrete and steel structure are shown in figures 6 and 7 respectively.

It is observed that for damage state with inter-story drift 2%, $ds,_2$, the median value of spectral acceleration calculated for complex concrete/steel frame is higher than the values proposed in Hazus methodology for both concrete and steel frame. The differences in median values are 35% and 24% for only steel and only concrete frame respectively.

In contrast, the standard deviation of spectral acceleration, for damage state with inter-story drift 2%, ds,2, calculated for complex concrete/steel frame is lower than the values proposed in Hazus methodology for both concrete and steel frame. The difference in standard deviation value is 20% which is the same for both steel and concrete frame respectively.

A comparison of fragility curves for concrete, steel and the two models of complex frame for damage state corresponding to 2% inter-story drift ratio, is presented in figure 8. It is proved that the complex frame with the current code is less vulnerable than all the others three types. The complex frame with the low code is more vulnerable than all the others three types. Additionally, a thorough observation is that concrete frame is slightly less vulnerable than the steel frame.

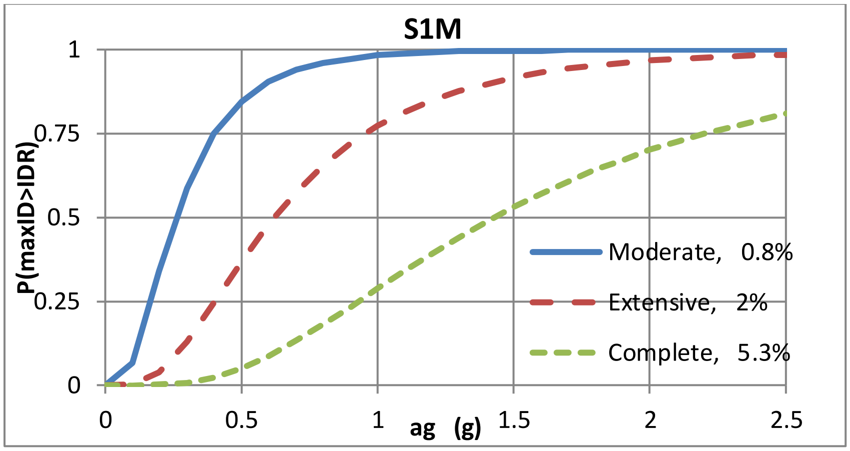

Figure 6 Fragility curves of steel structure according to Hazus approach

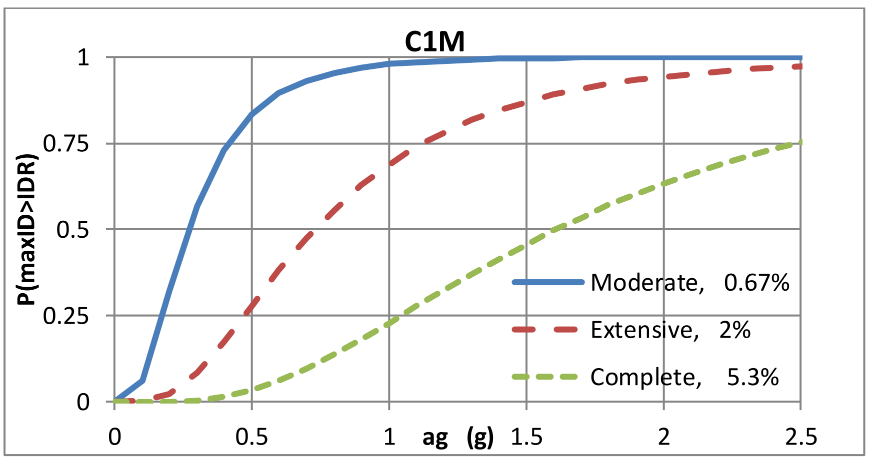

Figure 7 Fragility curves of concrete structure according to Hazus approach

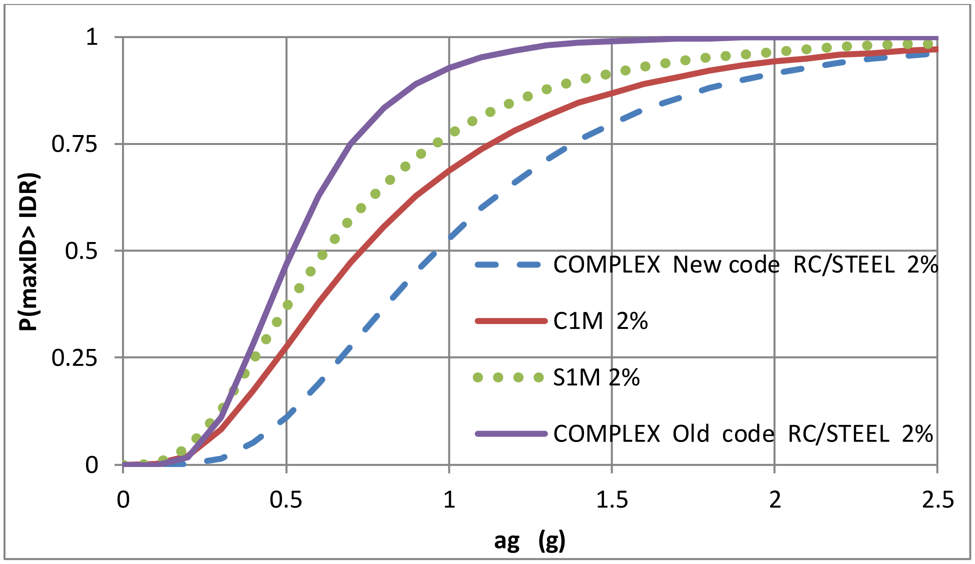

Figure 8 Fragility curves for concrete, steel and two models of complex frame for damage state corresponding to 2% inter-story drift ratio.

Conclusions

Fragility curves are an essential tool which direct deals with earthquake loss estimation. In this study, fragility curves for complex concrete/steel frames along their height, designed with different codes and for the three damage stages were developed. Complex concrete/steel frame consists of two parts, a lower one made by reinforced concrete and the upper one made by structural steel. The fragility curves obtained for such type of structures were compared with fragility curves for concrete or for steel only structures which are proposed in literature. The comparison shows that complex concrete/steel frame designed by current code is less vulnerable than the frame consists only of steel or only of concrete designed also with the new code. The results of this study can add to library of fragility curves for different type of structures. As rich is this library as better someone performs earthquake loss estimation analysis since now is capable to consider such a structural type in building population.

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