COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

INVESTIGATION INTO ANALYTICAL VULNERABILITY CURVES DERIVATION ASPECTS CONSIDERING MODELLING UNCERTAINTY FOR INFILLED RC BUILDINGS

A. Meslem[1], and D. D'Ayala[2]

[1] University College London
a.meslem@ucl.ac.uk

[2] University College London
d.d'ayala@ucl.ac.uk

Keywords: Fragility curves, Vulnerability Assessment, Modelling Uncertainty, Infilled RC building.

Abstract. *Reinforced concrete (RC) buildings represent the dominant type of construction in several earthquake prone countries and in the rest of the world. These structures are usually characterized by the presence of partition walls commonly made of unreinforced masonry bricks infill. To reduce the computational effort required to derive fragility functions, numerous researchers have attempted to use simplified assumptions, such as, the use of default values to model structural characteristics-related parameters (i.e., mechanical properties, geometric configuration, and dimensions), ignoring the contribution of infills in the seismic response by modelling infilled RC building as bare frame,...etc.,. However, such simplifications may highly decrease the reliability and accuracy of the obtained results introducing important epistemic uncertainty in the fragility function construction process. This paper investigates the different aspects of uncertainties resulting in adopting simplification in structural and mathematical modelling, and the resulting fragility curves. In the first section, the paper presents the results of sensitivity analysis that has been conducted examining the influence of the variation in structural characteristics-related parameters. The second section is devoted to a comparative analysis of different fragility curves developed with and without considering the contribution of infill panels (i.e. influence of numerical model completeness). The present study was conducted within the framework of the research project "Global Vulnerability Estimation Methods" funded by the Global Earthquake Model (GEM) foundation.*

1 INTRODUCTION

In order to reduce the calculation efforts, many researchers have attempted to use simplified assumptions in seismic vulnerability studies [28]. Indeed, it has been widely observed in literature [55, 31] that default nonlinear properties values (e.g. a default value of concrete strength, or steel strength, or an estimate of transverse reinforcement spacing...etc.), provided in existing guidelines/codes and implemented in commonly used structural programs, are assigned to represent mechanical, geometric configuration, and dimension characteristics. Usually, this is due either to lack of information, especially, for the case of older structures where design documents are generally not available, or to expedience.

Moreover, fragility curves of buildings located anywhere in the world have been generated using, for instance, HAZUS capacity curves derived for buildings in the US [36, 60]. This is particularly common when studies are conducted for large portions of the building stock and resources for direct survey and data acquisition are modest. Typically, differences in construction techniques and detailing between different countries are significant, even when buildings are nominally designed to the same code clauses. Furthermore, regarding the numerical modelling extensive literature review has shown that often vulnerability functions for infilled RC buildings are generated from analysis of bare frames structures [26, 32]. As a consequence, such assumptions and simplifications may highly decrease the reliability and accuracy of the obtained results due to the increase of epistemic uncertainty in modelling.

Apart from Liel et al (2008) very few studies have consistently analyzed the effect of the variability of several structural characteristics or of the simplified modelling assumptions on the generated fragility curves, with the scope of estimating the level of uncertainties that should be taken into account. In general practice, the aleatoric uncertainties associated to the structural characteristics-related parameters are accounted for by considering the probabilistic variability in their values [6, 8, 27, 29, 47, 50, 53, 54, 59]. In some others vulnerability studies [12, 13, 14, 33] the effect of dispersion in structural characteristics-related parameters were accounted for by survey of a large number of existing buildings and definition of a median and standard deviation of the sample of buildings, after calculation of the capacity and damage threshold for each building in the sample.

This paper presents the result of investigation on the sensitivity of fragility functions to variation in structural characteristics-related parameters' values and to numerical modelling completeness. The classes of structures considered are low-ductility RC buildings designed according to earlier seismic codes, and which are in general characterized by poor quality of materials, workmanship and detailing. To best identify the expected mean and range for the various parameter analysed a real frame in Turkey, is the reference prototype, however the methodology and results obtained are applicable to other typologies and locations, once the basic data is available. The sensitivity study is based on 3D nonlinear adaptive pushover analysis. The observations of the influence of variability of the selected parameters are conducted in terms of deformation capacity, considering different damage thresholds. The effect of model completeness is investigated by performing a comparative analysis of fragility curves derived with and without considering the contribution of masonry infill panels.

2 ADOPTED PROCEDURE

2.1 Selected capacity-related parameters

With regards to the effort by Liel and Deierlein [35], which has been also considered in the ATC-63 [3], the sensitivity study was conducted by considering height and framing system as variable parameters. In the present study, the investigated structural characteristics-related parameters are those associated to mechanical characteristics, dimension characteristics, and geometrical characteristics, as shown by Table 1. The mechanical characteristics parameters that have been considered are: compressive strength of concrete, yield strength of reinforcement, and strength of infill walls (in terms of compressive strength and thickness). Floor-to-floor story height was selected as dimensional parameter. In terms of geometrical characteristics, the examined associated parameter is transverse reinforcement spacing. The completeness of model is also taken into account by considering the effect of the contribution of infill walls. The structural characteristics-related parameters that have been considered are those associated to mechanical properties, geometric configuration, and structural details: compressive strength of concrete, yield strength of reinforcement, strength of infill walls (in terms of compressive strength and thickness), story height, and transverse reinforcement spacing. The reason for the choice of these parameters is to fill the gap regarding the availability of details on their effects with regard to the expected uncertainties that might result in deriving fragility functions. The choice of expected mean and range for each parameter, as shown in Table 1, is based on the results of structural characteristics assessment [7, 9], post-earthquakes surveys [18, 19, 21], on the requirement from different versions of earlier seismic codes, e.g. TS500 [58], and other references [22, 25, 30].

Parameters	Mean Value	Range of Value
Strength of reinforced concrete (fc)	17 MPa	14MPa ~ 20MPa
Tensile strength of steel (fy)	260 MPa	200MPa ~ 320MPa
Transverse reinforcement spacing (S)	200 mm	150mm ~ 250mm
floor-to-floor Story height (h)	2.8 MPa	2.5m ~ 3.2m
Thickness of infill walls (tw)	16 cm	13cm ~ 19cm
Compressive strength of infill walls (fw)	1.25 MPa	1.0MPa ~ 1.5MPa

Table 1: Expected mean and range of value for the structural characteristics-related parameters

2.2 Selected analysis type and numerical modelling

The accuracy of any selected procedure for the sensitivity analyses might depends on the type of the selected analysis approach, and the adopted mathematical model that must be consistent with the chosen numerical procedure. Different approaches, varying from nonlinear static to nonlinear dynamic analyses, might be used to conduct a sensitivity study, and estimate the performance of the structure. The sensitivity analyses conducted by Liel and Deierlein [35] were based on the implementation of Incremental Dynamic Analysis [61], using 2-D bare frame structures as numerical model.

Analysis type

In the present study, the sensitivity analyses have been based on the implementation of Adaptive Pushover Analysis, using 3-D bare frames and infilled frames structures as numerical models. One of the advantages in using this type of analysis is its relative simplicity in evaluating the response of inelastic structures, and can provide reasonable estimates of vulnerability functions and fragility curves. Furthermore, when using this type of analysis the variation in the structural stiffness at different deformation levels, and consequently the system degradation can be better accounted for [44].

	Model	Concrete Compressive Strength [MPa]	Steel Yield Strength [MPa]	Transverse Reinforcement Spacing [mm]	Story height [m]	Compressive Strength of Infill [MPa]	Thickness of Infill Panel [cm]
Infilled Frame Structures	M-01	**14**	260	200	2.8	16	1.25
	M-02	**15**	260	200	2.8	16	1.25
	M-03	**16**	260	200	2.8	16	1.25
	M-04	**17**	**260**	**200**	**2.8**	**16**	**1.25**
	M-05	**18**	260	200	2.8	16	1.25
	M-06	**19**	260	200	2.8	16	1.25
	M-07	**20**	260	200	2.8	16	1.25
	M-08	17	**200**	200	2.8	16	1.25
	M-09	17	**220**	200	2.8	16	1.25
	M-10	17	**240**	200	2.8	16	1.25
	M-11	17	**280**	200	2.8	16	1.25
	M-12	17	**300**	200	2.8	16	1.25
	M-13	17	**320**	200	2.8	16	1.25
	M-14	17	260	**150**	2.8	16	1.25
	M-15	17	260	**175**	2.8	16	1.25
	M-16	17	260	**225**	2.8	16	1.25
	M-17	17	260	**250**	2.8	16	1.25
	M-18	17	260	200	**2.5**	16	1.25
	M-19	17	260	200	**3.2**	16	1.25
	M-39	17	260	200	2.8	**13**	1.25
	M-40	17	260	200	2.8	**19**	1.25
	M-41	17	260	200	2.8	16	**1.00**
	M-42	17	260	200	2.8	16	**1.50**
Bare Frame Structure	M-20	**14**	260	200	2.8	---	---
	M-21	**15**	260	200	2.8	---	---
	M-22	**16**	260	200	2.8	---	---
	M-23	**17**	**260**	**200**	**2.8**	---	---
	M-24	**18**	260	200	2.8	---	---
	M-25	**19**	260	200	2.8	---	---
	M-26	**20**	260	200	2.8	---	---
	M-27	17	**200**	200	2.8	---	---
	M-28	17	**220**	200	2.8	---	---
	M-29	17	**240**	200	2.8	---	---
	M-30	17	**280**	200	2.8	---	---
	M-31	17	**300**	200	2.8	---	---
	M-32	17	**320**	200	2.8	---	---
	M-33	17	260	**150**	2.8	---	---
	M-34	17	260	**175**	2.8	---	---
	M-35	17	260	**225**	2.8	---	---
	M-36	17	260	**250**	2.8	---	---
	M-37	17	260	200	**2.5**	---	---
	M-38	16	220	100	**3.2**	---	---

Table 2: Infilled and Bare frames structures considered for the sensitivity analysis

Numerical modelling

A reinforced concrete member can be modeled with three constitutive material models: unconfined concrete (corresponding to the cover), confined concrete (corresponding to the core concrete) and reinforcing steel. Fiber-based structural modelling, which allows to discretize the cross section of the member to account for the different behaviour of cover and core concrete and steel, was adopted to model reinforced concrete members. By modelling separately the three constitutive material and their distribution over the cross-section the progression of nonlinear phenomena in the concrete member are more accurately simulated and hence one can capture more accurately response effects on such elements.

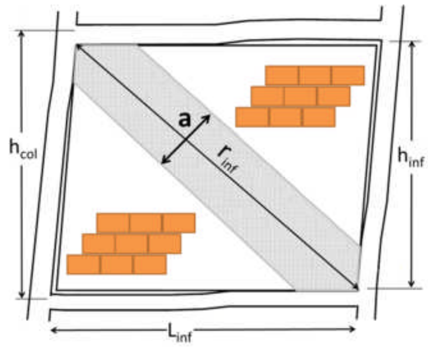


Figure 1: Equivalent diagonal strut representation of an infill panel

For infill panels, the buildings were modeled considering 50% of masonry infills [46]. Diagonal strut model, which has been the most frequently used by researchers and adopted in many documents and guidelines [1, 10, 40, 43], was implemented in this study (Figure 1). Each infill panel is simulated with a pair of compression struts. The equivalent strut "*a*" is computed using the formula based on the work of Mainstone and Weeks [39] and Mainstone [38]:

$$a = 0.175\left(\lambda_I\ h_{col}\right)^{-0.4} r_{\text{inf}} \quad , \quad \lambda_I = \left[\frac{E_m t_{inf} sin2\theta}{4E_f\ I_{col} h_{inf}}\right]^{\frac{1}{4}} \tag{1}$$

Global threshold damage states

With regard to the global threshold damage states, three global limit states: Slight Damage, Moderate Damage, and Near Collapse have been estimated from the analysis as a progression of local damage through several structural and non-structural elements. This is done by calculating for each element geometry and material characteristics, the ultimate concrete compressive strain [48], considering also the limit of curvature corresponding to the condition of yielding curvature and ultimate curvature, as indicated in Eurocode-8 [20] and FEMA-356 [2].

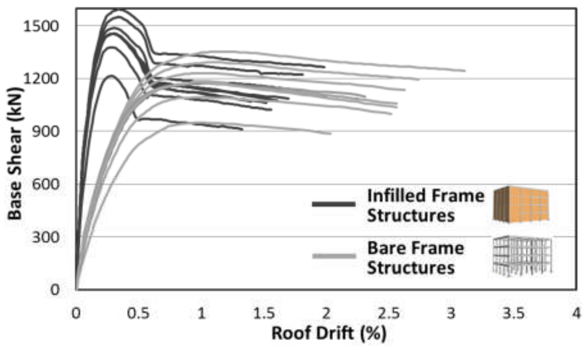


Figure 2: Resulted adaptive pushover curves for infilled frame and bare frame structures.

2.3 Selected models

Forty-two 3-D models, infilled frames and bare frames structures, were analyzed as shown in Table 2. For the considered structural characteristics-related parameters, the values were changed slightly representing the range the most probable for the case of existing low-ductility buildings constructed with respect to earlier seismic codes. The influence of these variations in the parameters' values has been observed in terms of deformation capacity for different damage conditions, obtained from adaptive pushover analysis. The response of each frame in terms of capacity curve is shown in Figure 2.

3 EFFECT OF STRUCTURAL CHARACTERISTICS-RELATED PARAMETERS

Figure 3 shows the influence of the variation in the structural characteristics-related parameters' values on the structure response, for different damage condition in terms of roof drift. Table 3 summarizes the level of sensitivity of the response to the change for each parameter in terms of Coefficient of Variation (CV), defined as ratio of standard deviation by mean value, and the percentage of difference (Diff) in deformation capacity for different damage condition.

The result of sensitivity analysis has shown that structural characteristics-related parameters are found to have a significant effect on the structural response, for different damage condition. Indeed, at the highest level of damage a remarkable variation (CV reaches a value up to 38%) in terms of deformation capacity (roof drift) has been observed even for a modest variation (CV = 12.7%) in compressive strength of concrete, as seen in Figure 3a (see Table 3); however, no difference in structure response has been found at the lowest level of damage, i.e. Slight Damage.

For tensile strength of steel, the effect has been found to be pretty different comparing to the compressive strength (Figure 3b). The effect is almost insignificant. For a CV= 16.1% of tensile strength, the CV in deformation capacity increase very slight-ly from Slight Damage to Moderate Damage and attained a value of only 9.2%, and then decrease to 7% at near collapse.

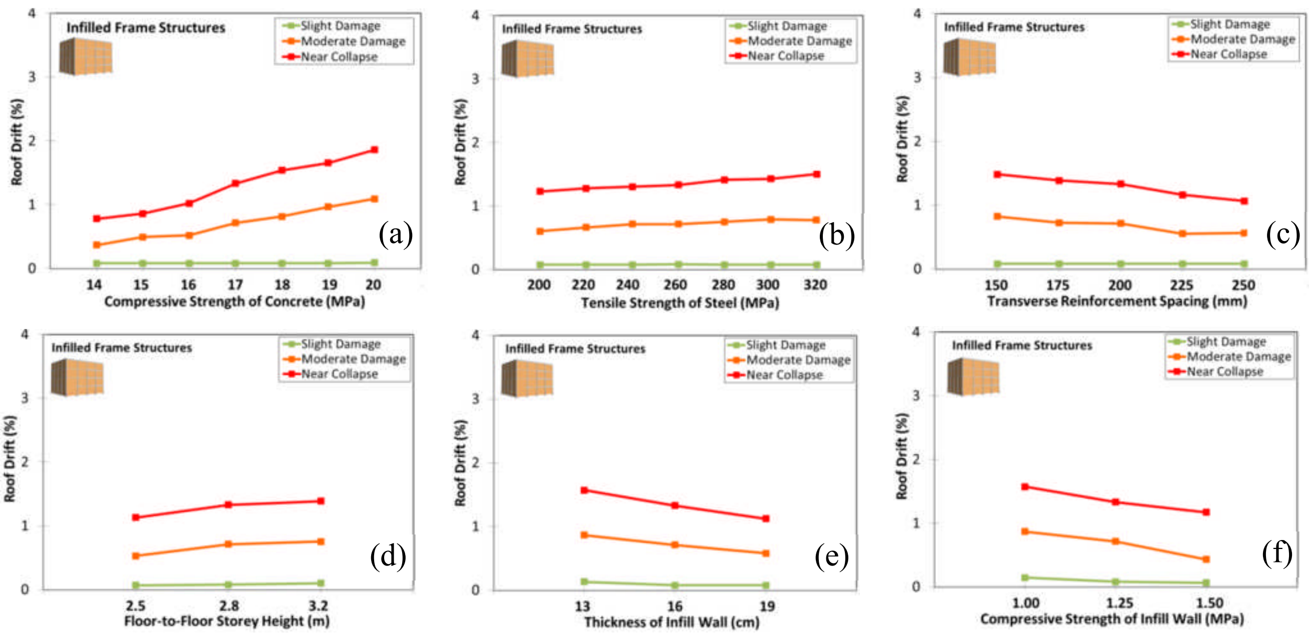


Figure 3: Sensitivity of the structure response to the variation in structural characteristics-related parameters' values. (a) Variation in compressive strength of concrete. (b) Variation in tensile strength of steel. (c) Variation in transverse reinforcement spacing. (d) Variation in storey height. (e) Variation in thickness of infill wall. (f) Variation in compressive strength of infill wall.

Parameters	Parameters' values		Slight Damage		Moderate Damage		Near Collapse	
	Range of Value	CV [%]	CV [%]	Diff. [%]	CV [%]	Diff. [%]	CV [%]	Diff. [%]
Strength of reinforced concrete	14MPa ~ 20MPa	12.71	4.13	11.11	37.47	197.56	32.28	139.08
Tensile strength of steel	200MPa ~ 320MPa	16.62	6.24	12.50	9.22	30.80	7.12	22.27
Transverse reinforcement spacing	150mm ~ 250mm	19.76	0.00	0.00	17.01	48.39	13.24	39.50
floor-to-floor Story height	2.5m ~ 3.2m	12.39	19.16	45.09	17.96	42.32	10.59	22.93
Thickness of infill walls	13cm ~ 19cm	18.75	31.49	66.67	19.85	49.23	16.65	39.68
Compressive strength of infill walls	1.0MPa ~ 1.5MPa	20.00	44.30	128.57	33.17	102.08	14.90	34.35

Table 3: Effect of the variation in the structural characteristics-related parameters' values on the structure response

In terms of ductility, the transverse reinforcement spacing was accounted for by adopting a certain range of values that are with respect to the result of structural characteristics assessment in existing buildings [19, 28]. Ac-cording to the result of sensitivity analysis, the structural response has been found to be moderately affected to the full range of variation in transversal reinforcement spacing (s=150 to 250mm), as shown in Figure 3c. For a variation of spacing CV = 19.76%, the CV in structural response attained a value of 18% and 10.6% for Moderate damage and near collapse, respectively. At Slight Damage level, no difference was observed in the structural response.

Floor-to-floor story height also shows a moderate effect on the seismic performance of the structure (see Figure 3d). The full range of variation CV = 12.4% leads to a remarkable difference in the de-formation capacity at different damage condition (CV in deformation capacity reaches value from 10.6 to 19.2%).

The effect of variation in the characteristics of in-fill walls was examined in terms of compressive strength and the thickness of infill walls. The sensitivity analysis was conducted for values range from 13cm to 19cm for thickness and 1.0MPa to 1.5MPa for compressive strength and of masonry infill walls, as shown in Figures 3e and 3f, respectively. According to the result of analyses, the two parameters have shown significance effect on the structure performance, at Slight to Moderate Damage condition. For the full range of variation of the thickness of infill walls, 18.75%, the structure response has been found to be CV=31.5% at Slight Damage and decrease to 19.85% at Moderate Damage. For CV=20% in the compressive strength of infill walls, the variation in structure response has been found to be CV=44.3% at Slight Damage and decrease to 33.2% at Moderate Damage. Both parameters show a significantly reduced effect on the structural response at Near Collapse. The variation in structure response has been found to be CV=16.65% for the full range of variation of compressive strength and 14.9% for the full range of variation of thickness of infills. This can be explained by the fact that the damage in infills in general occur at the early stage comparing to the RC members (see Figure 2, softening branch of the curve); hence, the infills will start to have less effect with the increase of damage.

Taking into consideration the class of buildings used in the analyses (which is low-ductility buildings characterized by poor quality of materials, workmanship and detailing), the different results shown above clearly reveal the significant effect that structural characteristics-related parameters variation might have on estimating structural response. It is to note that in the literature there have been some re-searchers who believe that the uncertainty in structural characteristics-related parameter such as mechanical properties, for instance, might be considered as not important as much as the uncertainty in the seismic record [45].

4 EFFECT OF NUMERICAL MODEL COMPLETNESS

Aiming to investigate the effect of numerical model completeness, a comparative analysis is performed between the two types of modelling frames systems, infilled frame with bare frame systems (Figure 4). For the same structural characteristics configuration, the infilled frame and bare frame models lead to a remarkable difference in estimating/capturing the response of the building (bias in deformation capacity), as shown in the Figure 4, and in Table 4. By considering the entire structural characteristics con-figuration, the computed mean value of deformation capacities from bare frame models has been found to be 6 times greater than the one calculated from in-filled frame models, at Slight Damage level. At the Moderate Damage level, the difference in the structural response between infilled frame and bare frame structures is also observed. The resulted mean value from bare frame models is found to be 2.2 times greater than the value from infilled frame models. For Near Collapse, this factor, in terms of mean value, is estimated to have a value of 1.8.

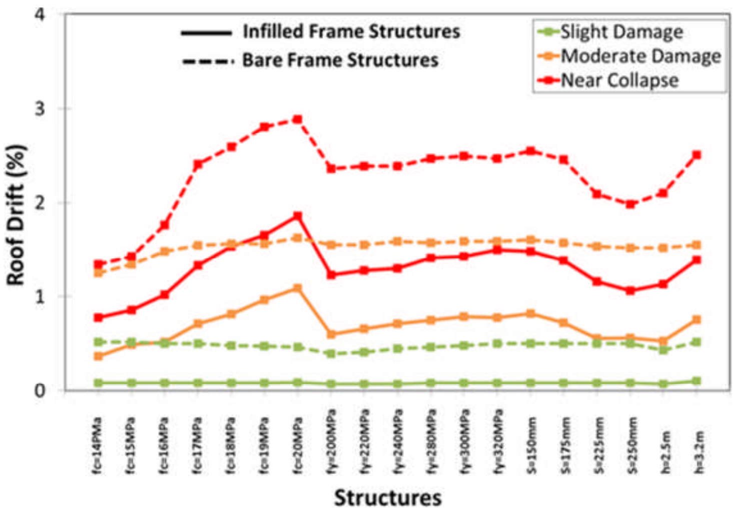


Figure 4: Comparison between the use of infilled frame and bare frame models for different structural characteristics configuration.

	Roof Drift					
	Slight Damage		Moderate Damage		Near Collapse	
	Mean [%]	CV [%]	Mean [%]	CV [%]	Mean [%]	CV [%]
Infilled Frame	0.08	8.71	0.69	24.88	1.30	20.35
Bare Frame	0.48	7.54	1.53	5.96	2.29	18.10
Factor	6		2.2		1.8	

Table 4: Sensitivity of structural response to the contribution of masonry infill panels.

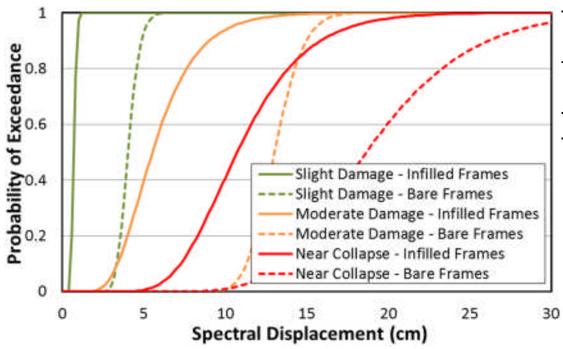
Regarding the above outcomes (i.e. the observed differences in deformation capacity between infilled frame and bare frame systems), it is worth to mention that modelling of infill is also associated to many other complex parameters that can be a source of significant uncertainty, such as, reduced strut width, strain at maximum stress, ultimate strain [41]. These parameters are in general calibrated directly from experiment. On the other hand, it should be recalled that in this study the masonry infills were modeled using equivalent diagonal strut model which is belonging to the macro model approach. In general, models from this approach are considered as simple, involving relatively modest calculations effort and computing time. However, there also numerous models which have been proposed in the literature and which are classified as belonging to the micro model approach. Models from this later approach are based on a finite element representation [24, 51, 52].

In order to derive fragility curves, the transformation of adaptive pushovers curves, defined in terms of base shear vs. top displacement, into capacity curves, defined in terms of pseudo spectral acceleration vs. spectral displacement, was carried out using the standard approach documented in many codes of practice, e.g. ATC-40 [4]; HAZUS-MH MR3 [23]. Fragility curves have been derived by selecting thirteen (13) infilled frame models, and thirteen (13) bare frame models (see Figure 4 for the selected models). Assuming a lognormal distribution (common assumption in seismic studies), fragility curves are defined as the conditional probability of being in or exceeding, a particular damage state ds_i, given the spectral displacement, S_d:

$$P\left[ds \geq ds_i \ |S_d\right] = \Phi\left[\frac{1}{\beta_{ds_i}} ln\left(\frac{S_d}{\overline{S}_{d,ds_i}}\right)\right] \quad (3)$$

where, $\overline{S}_{d,ds_i}$ is the median value of spectral displacement at which the building reaches the threshold of damage state ds_i ; β_{ds_i} is the standard deviation of the natural logarithm of spectral displacement for damage state ds_i ; Φ is the standard normal cumulative distribution function.

Using the aforementioned procedure, Figure 5 shows the fragility curves that have been derived for each system, i.e., infilled frame and bare frame structures. Each fragility curve is defined by a median value of spectral displacement that corresponds to the threshold of that damage state, and by the variability (β) associated with that damage state. These two parameters are estimated using the First-Order Second-Moment (FOSM) method. Detailed information about the models can be found in D'Ayala and Meslem [63].



System	Slight Damage		Moderate Damage		Near Collapse	
	Median [mm]	β	Median [mm]	β	Median [mm]	β
Infilled Frames	7	0.18	55	0.38	107	0.30
Bare Frames	40	0.15	130	0.11	187	0.26
Factor	5.8		2.4		1.7	

Figure 5: Comparisons of fragility curves of the structures with and without considering the contribution of masonry infill walls.

It is evident form Figure 5 the role played by the inclusion or exclusion of the masonry infill in the modelling. The exclusion of infills' contribution leads to a significant bias in fragility curves. The median capacity varies by a factor of 5.8, 2.4, and 1.7 for Slight Damage, Moderate Damage, and Near Collapse, respectively. Moreover, when the infilled RC building is modeled as bare frame structure, the resulted fragility curves show greater lateral displacement capacity for all damage levels; whereas, the building is found to be more vulnerable when the infilled frame model is used. Indeed, the result of pushover analysis has shown (either for infilled frame or bare frame structures) that the first-storey mechanism was the most observed, and the presence of infills leads to the occurrence of this mechanism at earlier stage comparing to the case of bare frame structures. It should be noted that difficulties have been encountered to predict shear failure which is still not fully understood despite much experimental research and analysis. In fact, the shear column failure might have a significant effect on the structure performance, especially for infilled building designed without considering horizontal actions, or building with low concrete strength.

With regards to the dispersion, β, a simplification has been used in the literature in the expression of the uncertainty in the structure capacity. For instance, Kappos et al. [30] constructed fragility curves by adopting value of 0.3 and 0.25 for the uncertainty in the capacity for low and high code buildings, respectively, assuming that these values are for all damage states. These values have been suggested by FEMA-NIBS [23]. Throughout the study by Shahzada et al. [57] a same value of 0.3 was assigned for the uncertainty associated with the capacity curve of buildings for all damage states, as it is proposed in Wen et al. [62]. Satter and Liel [56] and Raghunandan et al. [49] have used a value of 0.5, which has been suggested based on previous research work by Liel et al. [34], to account for uncertainty due to the structural modelling, for Collapse level only. However, the results of present study have clearly shown that the value of dispersion, β, is not the same for all damage states and that the trends from one state of damage to next not necessarily linear, and not systematic. For the case of building that has been investigated in this study, the value of β is found to be bigger at the Moderate Damage for the case of infilled frame model (β = 0.38), while for bare frame the biggest value is found to be at Near Collapse (β = 0.26). On the other hand, β is found to be smaller at the Slight Damage for the two modelling option (β = 0.18 for infilled frame, and 0.15 for bare frame model).

5 CONCLUSIONS

- This present study was devoted to examine the influence in modelling the building capacity-related parameters in aim to probe the issue of their associated uncertainty in predicting the seismic performance and derivation of fragility functions.
- It was clearly observed that special care should be given when assigning values to represent the structural details, especially, material characteristics-related values. Reinforced concrete strength-related variation values have shown a significant effect on the building capacity, and this effect increase with the progress of damage condition.
- The comparison of fragility curves between the two modelling types, infilled and bare frames, indicate a significant difference in predicting the seismic performance of the building. Modelling building as bare frame structure lead to lowest risk of damage, however, the building is found to be more vulnerable if the infilled frame system is adopted.
- Trends in dispersion from one state of damage to next and from one modelling option to next might not necessarily be linear, and monotonic.
- For the masonry infills, the two parameters considered in the sensitivity analysis are compressive strength and thickness of infills. However, this element is also associated to many other complex parameters that can be a source of significant uncertainty, such as,

reduced strut width, strain at maximum stress, ultimate strain. These parameters are in general calibrated directly from experiment.

- Difficulties that might be encountered to predict shear failure which is still not fully understood despite much experimental research and analysis.
- The sensitivity analyses was conducted for low-ductility RC buildings designed according to earlier seismic codes and which are, in general, characterized by poor quality of material. Hence, more investigation should be conducted for building with high compressive strength in aim to analyse the sensitivity of different capacity-related parameters on seismic performance prediction and derivation of vulnerability curves.

REFERENCES

[1] American Society of Civil Engineers (ASCE), *Seismic rehabilitation of existing buildings.* ASCE Standard ASCE/SEI 41-06, Southpointe, Canonsburg, PA., 2006.

[2] American Society of Civil Engineers (ASCE), *FEMA-356: Prestandard and commentary for the seismic rehabilitation of buildings.* Federal Emergency Management Agency, Washington, DC, 2000.

[3] Applied Technology Council (ATC), *Recommended methodology for quantification of building system performance and response parameters*, ATC-63 90% Draft. Redwood City, CA, 2007.

[4] Applied Technology Council (ATC), *Seismic evaluation and retrofit of concrete buildings*, ATC-40. Redwood City, CA, 1996.

[5] Applied Technology Council (ATC), *Seismic performance assessment of buildings, Volume 1 - Methodology*, ATC-58-1 75% Draft. Redwood City, CA, 2011.

[6] B.O. Ay, and A. Erberik, Vulnerability of Turkish low-rise and mid-rise reinforced concrete frame structures. *Journal of Earthquake Engineering*, 12, 2-11, 2008.

[7] B.O. Ay, *Fragility based assessment of low-rise and mid-rise reinforced concrete frame buildings in Turkey*, Middle East Technical University, MSc. Thesis, 2006.

[8] A. Bakhshi, and K. Karimi, Method of developing fragility curves a case study for seismic assessment of masonry buildings in Iran. *7th International Congress in Civil Engineering*, Tehran, Iran, 2006.

[9] I.E. Bal, H. Crowley, and R. Pinho, Detail assessment of structural characteristics of Turkish RC buildings stock for loss assessment models. Soil Dynamic and Earthquake Engineering, 28, 914-932, 2008.

[10] Canadian Standards Association (CSA), *Design of masonry structures CSA-S304.1-04*, Ontario, Canada, 2004.

[11] F. Colangelo, Pseudo-dynamic seismic response of reinforced concrete frames in lled with non-structural brick masonry. *Earthquake Engineering and Structural Dynamics*, 34,1219-1241, 2005.

[12] D.F. D'Ayala, Force and displacement based vulnerability assessment for traditional buildings. *Bulletin of Earthquake Engineering*, 3, 235-265, 2005.

[13] D.F. D'Ayala, R. Spence, C. Oliveira, and A. Pomonis, Earthquake loss estimation for Europe's historic town centres. *Earthquake Spectra*, 13, 773-793, 1997.

[14] D.F. D'Ayala, and S. Paganoni, Assessment and analysis of damage in L'Aquila historic city center after 6th April 2009. *Bulletin of Earthquake Engineering*, 9, 81-104, 2010.

[15] M. Dolsek, Incremental dynamic analysis with consideration of modelling uncertainties, *Earthquake Engineering and Structural Dynamics*, 38, 805-825, 2009.

[16] M. Dolsek, and P. Fajfar, The effect of masonry infills on the seismic response of a four storey reinforced concrete frame-a probabilistic assessment, *Engineering Structures*, 30, 1991-2001.

[17] C. Dymiotis, A. Kappos, and M. Chryssanthopoulos, Seismic reliability of masonry-infilled rc frames. *Journal of Structural Engineering*, 127, 296-305.

[18] Earthquake Engineering Field Investigation Team (EEFIT), *The Kocaeli, Turkey earthquake of 17 August 1999*. Institution of Structural Engineers, a Field Report by EEFIT, 2003.

[19] Earthquake Engineering Research Institute (EERI), *Kocaeli. Tyurkey, earthquake of August 17, 1999: reconnaissance report*. Earthquake Spectra, 2000.

[20] European Committee for Standardization (CEN), *Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings*, Eurocode-8, ENV 1998-1-1, Brussels, Belgium, 2004.

[21] F.L. Ellul, *Static nonlinear finite element analysis of low engineered masonry infilled reinforced concrete frames for seismic assessment*, PhD Thesis, University of Bath, 2006.

[22] G. Erol, E. Yuksel, H. Saruhan, G. Sagbas, P.T. Tuga, and H.F. Karadogan, A complementary experimental work on brittle portioning walls and strengthening by carbon fibers. Proceeding of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, 2004.

[23] Federal Emergency Management Agency – National Institute of Building Sciences (FEMA-NIBS), Multi-hazard loss estimation methodology – earthquake model: HAZUS®MH Advanced engineering building module - Technical and user's manual, Washington DC., 2003.

[24] A.K. Ghosh, and A.M. Made, Finite element analysis of infilled frames. *Journal of Structural Engineering*, 128, 881-889, 2002.

[25] P. Gulkan, M. Aschhein, and R. Spence, *Reinforced concrete frame building with masonry infills*, World Housing Encyclopedia, Report No. 64, 2002.

[26] H.A. Howary, and S.S.F. Mehanny, Seismic vulnerability evaluation of RC moment frame buildings in moderate seismic zones. *Earthquake Engineering and Structural Dynamics*, 40, 215-235, 2011.

[27] I. Iervolino, G. Manfredi, M. Polese, G.M. Verderame, and G. Fabbrocino, Seismic risk of RC building classes. *Engineering Structures*, 29, 813-820, 2007.

[28] M. Inel, and H.B. Ozmen, Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. *Engineering Structures*, 28, 1494-1502, 2006.

[29] H. Jiang, H. Lu, and L. Chen, Seismic fragility assessment of RC moment-resisting frames designed according to the current Chinese seismic design code. *Journal of Asian Architecture and Building Engineering*, 11, 153-160, 2012.

[30] A.J. Kappos, G. Panagopoulos, and G. Penelis, A hybrid method for the vulnerability assessment of R/C and URM buildings. *Bulletin of Earthquake Engineering*, 4 , 391-413, 2006.

[31] R.A. Khan, and T. Naqvi, Reliability analysis of steel building frame under earthquake forces. *International Journal of Emerging Technology and Advanced Engineering*, 2, 356-361, 2012.

[32] M.S. Kircil, and Z. Polat, Fragility analysis of mid-rise R/C frame buildings. *Engineering Structures*, 28, 1335-1345, 2006.

[33] K. Lang, and H. Bachmann, On the seismic vulnerability of existing buildings: A case study of the city of Basel. *Earthquake Spectra*, 20, 43-66, 2004.

[34] A.B. Liel, C.B. Haselton, G.G. Deierlein, J.W. Baker, Incorporating modelling uncertainties in the assessment of seismic collapse risk of buildings. *Structural Safety*, 31, 197-211, 2009.

[35] A.B. Liel, and G.G. Deierlein, *Assessment the collapse risk of California's existing reinforced concrete frame structures: Metrics for seismic safety decisions*. Blume Center Technical Report No. 166, 2008.

[36] L.A. Lourdes, R.R. Lopez, A. Saffar, Development of fragility curves for medium rise reinforced concrete shear wall residential buildings in Puerto Rico. S.A. Elaskar, E.A. Pilotta, G.A. Torres eds. *Mecanica Computacional Vol XXVI,* Cordoba, Argentina, October, 2007.

[37] A. Madan, and A. Hashmi, Analytical prediction of the seismic performance of masonry infilled reinforced concrete frames subjected to near-field earthquakes. *Journal of Structural Engineering*, 134, 1569-1581.

[38] R.J, Mainstone, *On the stiffness and strengths of infilled frame*. Proceedings Institution of Civil Engineers, Supplement IV, 57–90, 1971.

[39] R.J. Mainstone, G.A. and Weeks, *The influence of bounding frame on the racking stiffness and strength of brick walls*. 2nd International Brick Masonry Conference, Stoke-on-Trent, UK, 1970.

[40] Masonry Standards Joint Committee (MSJC), *Building code requirements for masonry structures, ACI 530-02/ASCE 5-02/TMS 402-02*, American Concrete Institute, Structural Engineering Institute of the American Society of Civil Engineers, The Masonry Society, Detroit, 2002.

[41] A. Meslem, and D. D'Ayala, *Toward worldwide guidelines for the development of analytical vulnerability functions and fragility curves at regional level*. 15th World Conference on Earthquake Engineering, Lisbon, Portugal, 2012.

[42] G.V. Mulgund, and A.B. Kulkarni, Seismic assessment of rc frame buildings with brick masonry infills. *International Journal of Advanced Engineering Sciences and Technologies*, 2, 140-147, 2011.

[43] New Zealand Society for Earthquake Engineering (NZSEE), *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, 2006.

[44] V.K. Papanikolaou, and A.S. Elnashai, Evaluation of conventional and adaptive pushover analysis I: methodology. *Journal of Earthquake Engineering*, 10, 923-941, 2005.

[45] L. Pasticier, C. Amadio, and M. Fragiacomo, Non-linear seismic analysis and vulnerability evaluation of a masonry building by means of the SAP2000 V.10 code. *Earthquake Engineering and Structural Dynamics*, 37, 467-485, 2008.

[46] T. Paulay, and M.J.N. Priestley, *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons, Inc, 1992.

[47] M. Polese, G.M. Verderame, C. Mariniello, I. Iervolino, and G. Manferedi, Vulnerability analysis for gravity load designed RC buildings in Naples – Italy. *Journal of Earthquake Engineering*, 12, 234-245, 2008.

[48] M.J.N. Priestley, F. Seible, and G.M.S. Calvi, *Seismic design and retrofit of bridges*. New York: John Wiley & Sons, 1996.

[49] M. Raghunandan, A.B. Liel, H. Ryu, N. Luco, and S.R. Uma, *Aftershock fragility curves and tagging assessment for a mainshock-damaged building*. 15th World Conference on Earthquake Engineering, Lisbon, Portugal, 2012.

[50] P. Rajeev and S. Tesfamariam, Effect of construction quality variability on seismic fragility of reinforced concrete building. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering Building and Earthquake-Resilient Society*, Auckland, New Zealand, April 14-16, 2011.

[51] J.R. Riddington, and B. Stafford-Smith, Analysis of infilled frames subject to racking with design recommendations. *The Structural Engineer*, 55, 263-268, 1977.

[52] C.E. Rivero, and W.H. Walker, *An analytical study on the interaction of frames and infill masonry walls*. 8th World Conference on Earthquake Engineering, Proceddings, Vol. IV, 591-598, 1984.

[53] T. Rossetto, and A. Elnashai, A new analytical procedure for the derivation of displacement-based vulnerability curves for populations of RC structures. *Engineering Structures*, 27, 397-409, 2005.

[54] H. Ryu, N. Luco, H.R. Uma, and A.B. Liel, Developing fragilities for mainshock-damaged structures through incremental dynamic analysis. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering Building and Earthquake-Resilient Society*, Auckland, New Zealand, April 14-16, 2011.

[55] S. Salvador, A. Monica, S. Michael, J. Cristian, and E. Luis, Seismic response of reinforced concrete buildings in Caracas, Venezuela. *The 14th World Conference on Earthquake Engineering*, Beijin, China, October 12-17, 2008.

[56] S. Sattar, and A.B. Liel, *Seismic performance of reinforced concrete frame structures with and without masonry infill walls*. 9th U.S. National and 10th Canadian Conference on Earthquake Engineering, Toronto, Canada, 2010.

[57] K. Shahzada, B. Gencturk, A.N. Khan, A. Naseer, M. Javed, and M. Fahad, Vulnerability assessment of typical buildings in Pakistan. *International Journal of Earth Sciences and Engineering*, 4, 208-211, 2011.

[58] Turkish Standards Institute (TSE), *TS-500, Building code requirements for reinforced concrete*. Ankara, Turkey, 1985.

[59] S.R. Uma, H. Ryu, N. Luco, A.B. Liel, and M. Raghunandan, Comparison of main-shock and aftershock fragility curves developed for New Zealand and US buildings. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering Building and Earthquake-Resilient Society*, Auckland, New Zealand, April 14-16, 2011.

[60] R. Vacareanu, A.B. Chesca, B. Georgescu, M. Seki, Case study on the expected seismic losses of soft and weak groundfloor buildings. *International Symposium on Strong Vrancea Earthquake and Risk Mitigation*, Bucharest, Romania, October, 2007.

[61] D. Vamvatsikos, and C.A. Cornell, Incremental dynamic *analysis. Earthquake Engineering and Structural Dynamics*, 3, 491-514, 2002.

[62] Y.K. Wen, B.R. Ellingwood, and J. Bracci, *Vulnerability function framework for consequence-based engineering*. Mid-America Earthquake (MAE) Center, Project DS-4 Report, 2004.

[63] D'Ayala, D. and Meslem. A. Derivation of analytical vulnerability functions considering modeling uncertainties. *Proceeding of the 11th International Conference on Structural Safety & Reliability*, New York, 2013.