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A CRITICAL REVIEW OF FRAGILITY CURVES FOR EXISTING RC BUILDINGS

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Abstract. *In recent years, many studies were been carried out in order to understanding and later mitigate the seismic risk. Several important projects have been funded, different methodologies have been developed; in particular, for emergencies management several application have been carried out with good results. On the contrary, mitigation and prevention of seismic risk could be more efficient by setting a careful assessment, maintenance and retrofitting of the built. In this sense, it to be noted that the seismic capacities of existing RC buildings have shown a key role in recent seismic events (e.g. Southern Italy 1980, Turkey 1999, L'Aquila 2009, Lorca, 2011, Emilia plan 2012). In particular, old RC buildings have often shown a poor and brittle behavior. Moreover, the low seismic performances of these buildings are the main reason of significant earthquake losses (in terms of economic, social and political activities) that can been considered generally as a direct consequence of physical damages on the buildings. About these important topics, it is the opinion of the authors, that quantitative models of fragility, referring to the most common types of buildings, have a key role in the evaluation process of risk and should been continuously improved. Therefore, in the seismic risk studies, a fundamental step is the development and use of fragility curves representative of the behavior of existing RC buildings. A significant number of proposals are currently available in the scientific literature. In this study, a critical review of existing different procedures for RC with Moment Resisting Frames (MRF) has carried out in order to highlight advantages and weakness of each proposal. A great variability in terms of geometrical, mechanical and structural characterization, structural modeling, method of analysis, scale of damage, parameters of seismic intensity and statistical procedure has been highlighted, and finally an optimal procedure of fragility analysis has been outlined.*

1 INTRODUCTION

Recent European earthquakes have shown that the economic loss and urban resilience are closely related to the seismic performance of existing buildings, designed without seismic criteria or with old codes, that showed an unsatisfactory behavior [1]. For this reason, in order to mitigate the seismic risk and increase resilience in urban areas, reduction strategies risk should be developed.

In seismic risk mitigation polices, with regard to the most common types of existing buildings the quantitative fragility models have a key role. Exist different approaches for the construction of fragility models: analytical approaches, empirical approaches, approaches based on expert opinion and hybrid method. In this paper, only analytical methods have been considered; they are based on damage distributions simulated from the numerical analyses. Due to the importance of the topic, a significant number of studies were developed and published in the last years. In first part of the paper, a short critical review of different methods and procedures has been performed. In this way, the difference due to choices about analysis method, idealization, seismic hazard, and the damage model, have been highlighted. In particular, following a careful literature review, six studies have been selected, with the same analysis object and purpose. In fact, the selected seismic risk studies , have investigated the existing Reinforced Concrete (RC) with Moment Resisting Frame (MRF). These typologies represent the highest percentage of building stock in several European areas with high seismicity, and they have similar properties.

For each study the main advantages and weakness have been pointed out. Then, the paper focuses on the importance of an adequate damage model; damage model should be able to take in to account the different damage state, as structural and non structural damage. The main topics of different damage models considered have been highlighted.

2 CRITICAL REVIEW OF METHODS

A wide literature review have been carried and six studies have been selected; they are focused on the construction of fragility curves and resulting FCs have been defined for similar structural typologies of existing Reinforced Concrete with Moment Resisting Frame structures (RC-MRF).

The selected studies are following listed and in the following they will be called with relative acronym: Masi et Vona 2012 [2] (MV12), Vona 2014 [3] (V14), Kyriakides et al. 2015 [4] (K15), Polese et al. 2008 [5] (P08), Erberik 2008 [6] (E08) and Silva et al. 2014 [7] (S14). In this studies, different methodologies have been defined in order to obtain fragility curves. In table 1 are reported the main characteristics of each study.

Several main steps can be identified in all studies:

- Selection of the structural types most vulnerable and widespread in region under examination;
- Geometrical, structural and mechanical characterization of selected structural class;
- Generation of adequate sample of structural models able to represent the real geometrical, structural and mechanical variability;
- Selection of ground motion;
- Structural modeling and evaluation of the seismic response;
- Definition of damage model;
- Construction of fragility curves.

In each step, a probabilistic approach should be used, because a high degree of uncertainty involved each of them. The structural models for numerical simulation should be able to represent the real probabilistic variability (for example material and geometric-structural, properties of structural, ect).

In several cases, it is excessive consider probabilistic some topics, especially when their variation is negligible, and not affect substantially the structural behavior. Farther, a probabilistic approach requires an accurate and extensive investigation in order to obtain reliable probability distributions. Actually, the highest computational power and the available numerical method allow a significant diffusion of the probabilistic approaches.

	Framework	V14	K15	P08	E08	S14
Selection and characterization of reference structural class	Building type studied	RC-MRF				
	Area surveyed	Italy and Mediterranean countries	Limassol (Cyprus)	Arenella district Naples	Dűzce (Turkey)	Marmara region (Turkey)
Approach used in geometrical-mechanical-structural characterization of sample RC-MRF D=Deterministic P=Probabilistic	Form in plan	D	D	D	D	D
	Dimension plan	D	D	P	D	D
	Interstorey height	D	D	D	D	P
	Number of storey	D	D	D	D	D
	Beam length	D	D	P	D	P
	Column depth	D	D	D	D	P
	Concrete strength	D	P	P	D	P
	Steel yield strength	D	P	P	D	P
Evaluation of seismic response	Analysis Method	NLDA	NLDA	NLSA	NLDA	NLSA and NLDA
	Structural modeling	2D lumped plasticity	2D fiber element	3D lumped plasticity	SDOF	2D fiber element
	Type of seismic action	natural accelerogram	accelerogram base on spectrum	accelerogram base on code spectrum	natural accelerogram	natural accelerogram
Construction of FC	Intensity parameter	I_H	S_d	S_d	PGV	S_a
	Probability distribution	lognormal	lognormal	lognormal	lognormal	lognormal

Table 1: Main characteristics of frameworks selected.

In any cases, the trivial uses of probabilistic approaches carry out to unnecessary or incorrect choice about structural models or unrepresentative characteristics (for example, for beam length and column depth). Therefore, several simplified approaches (for example MV12/V14/E08) that consider any probabilistic variables as deterministic are more reliable. In other words, RC-MRF structural models are able to reproduce the behavior of real buildings also choices several deterministic values. For example, in MV12, V14 and E08, the role of infill masonry walls have been considered; they have been investigated with deterministic approach.

Generally, reliable FCs should be defined on the based of more accurate NonLinear Dynamic Analyses result (as in MV12, V14, K15, S14). On the contrary, FCs based on NonLinear Static Analyses (P08/S14) could be less able to simulate the real behavior of buildings. In E08, FCs have been defined on NLDAs; nevertheless the global response of buildings have been investigated using SDOF equivalent analytical models that have been characterized from structural non linear static analysis.

The seismic input plays a key role in FS definition. In order to obtained a realistic evaluation of structural performance, the accelerograms recorded during real earthquakes should be considered further less appropriate synthetic events. In V14, S14, E08 natural accelerograms extracted from different data-base have been used; in K15, time-history have been derived from response spectrum; finally, in P08, EC8 [10] spectrum has been used.

Equally relevant, the seismic intensity should be able to represent the damage potential of ground motion. Integral seismic parameters, such as Arias Intensity I_A and Housner Intensity I_H seem more effective with regard to peak or spectral parameters [11].

2.1 Characterization of Damage Models

In FCs definition, the Damage Model plays a key role. Damage Model should be defined from limit states that define the thresholds between different damage conditions. The limit states should be able to take into account the structural e nonstructural damage and their evaluation. Further, for each limit state should be associate an analytical characterization using a Damage Measure.

The main distinction in terms of Damage Measure (DM) is local or global DM. The first is structural response parameters due to single structural members; the second is referred to whole structure. The choice of local or global DM is strongly linked with modeling and analysis methods choices. For example, if the equivalent SDOF are considered, the limit states cannot be defined in a detailed way (e.g. based on member behavior, local strains or hinge mechanisms, ecc). In these cases, the global DM will be defined in terms of simplified global parameters. In addition, for each limit state a qualitative description of non-structural e structural damage should be considered (for example, using a typical damage scale as EMS98 [8] or specific defined scale). In the tables 2-3-4, the damage models used in the investigated studies are reported.

Damage model (V14)			**Damage model (MV12)**
EMS98	**Damage Level**	**Limit condition**	**Limit condition**
0	SD=None; NSD=None	IDR $\leq$ 0,05%	IDR$\leq$ 0,1%
1	SD=None; NSD=Weak	$R_y \leq 1$ and 0,05%<IDR$\leq$0,1%	0,1% < IDR$\leq$ 0,25%
2	SD=Low; NSD=Moderate	0<$R_p \leq$0,25 or IDR>0,1% and $R_y \leq 1$	0,25% < IDR $\leq$ 0,5%
3	SD=Medium; NSD=Significant	0,25< $R_p \leq$0,75	0,5% < IDR$\leq$1%
4(5)	Near Collapse/Collapse	0,75 < $R_p \leq$ 1	IDR>1%
SD=Structural Damage; NSD=NoStructural Damage $R_y=\phi/\phi_y$; $R_y=\phi-\phi_y/\phi_u-\phi_y$			

Table 2: Damage Model in V14 and MV12.

Damage model (E08)		**Damage model (S14)**	
Limit state	**Limit condition**	**Limit state**	**Limit condition**
Serviceability LS	S_I=0.2	Limit state 1	IDR(%)→Δ_{roof}→75% $V_{base,max}$
Damage Control LS	IDR(%)→$\Delta_{roof,DC}$=75% $\Delta_{roof,CP}$	Limit state 2	IDR(%)→Δ_{roof}→$V_{base,max}$
Collapse Prevention LS	IDR(%)→$\Delta_{roof,CP}$=75% $\Delta_{roof,max}$	Limit state 3	IDR(%)→Δ_{roof}→$V_{base,max}$ descrease of 20%
S_I=Softening index			

Table 3: Damage Model in E08 and S14.

Damage model (K15)	
Limit state	**Limit condition**
Damage Limitation	$\theta_{column} < \theta_y$
Significant Damage	$\theta_{column} < 3/4\theta_u$
Near Collapse	$\theta_{column} = \theta_u$ and $V=V_R$
Building Collapse	all columns of floor reach L.S.3 or IDR=4%

Table 4: Damage Model in K15.

Generally, the interstorey drift is considered as Damage Measure. The interstorey drift is a good damage index for RC-MRC structures but reliable specific values should be defined. In this way, some results could be used: real test building during several earthquakes; laboratory experimental dynamic and pseudo-dynamic tests on models in scale or in full-scale; virtual experimental tests using numerical simulation.

Experimental calibration and validation of interstory drift limit is an hard work [9]. Generally, specific values should be defined in each studies and projects. In S14 and E08 each limit state has been defined through specific interstorey drift value; these values are corresponding to limit state in terms of base shear and roof displacement.

In order to verify the representativeness of these condition, these values have been compared with the interstory drift as defined in V14. The interstory drift limit values for each classes in according to E08 and S14 are reported in Table 5 and Table 6.

DM S14	IDR%(BF 2storey Pre71)	IDR%(IF 2storey Pre71)	IDR%(PF 2storey Pre71)
LS1	0,48	0,07	0,32
LS2	0,80	0,33	0,71
	IDR%(BF 4storey Pre71)	IDR%(IF 4storey Pre71)	IDR%(PF 4storey Pre71)
LS1	0,66	0,08	0,26
LS2	0,89	0,41	0,58
	IDR%(BF 8storey Pre71)	IDR%(IF 8storey Pre71)	IDR%(PF 8storey Pre71)
LS1	0,32	0,11	0,11
LS2	0,83	0,49	0,52

Table 5: Inter-story drift values for LS1 and LS2 of DM S14.

E08	IDR% (BF 2storey Pre71)	IDR% (IF 2storey Pre71)	IDR% (PF 2storey Pre71)
Serviceability LS	0,32	0,20	0,44
Damage Control LS	0,84	0,50	0,74
Prevention Collapse LS	1,17	0,67	1,01
	IDR% (BF 4storey Pre71)	IDR% (IF 4storey Pre71)	IDR% (PF 4storey Pre71)
Serviceability LS	0,22	0,11	0,19
Damage Control LS	0,75	0,50	0,60
Prevention Collapse LS	0,94	0,67	0,77
	IDR% (BF 8storey Pre71)	IDR% (IF 8storey Pre71)	IDR% (PF 8storey Pre71)
Serviceability LS	0,53	0,15	0,22
Damage Control LS	1,19	0,70	0,66
Prevention Collapse LS	1,57	0,93	0,91

Table 6: Inter-story drift values for Limit States of DM E08

On the basis of the values reported in Table 5 and 6, it must highlighted that LS and LS2 is generally equal, except for structures high-rise types buildings. Generally, the limit LS3 is not able to represent the limit between extensive damage and structural collapse. The values reported in Table 5-6 have been compared with interstory drift (IDR) values defined from V14. In this work, the comparison between frameworks reported described in table 1 has been carried out in a graphic way (figures 1 - 6) for 2 storey Bare Frame Pre71 type buildings.

As main results, the global limit condition used by S14 and E08 are not consistent with local condition of V14; in particular, it to be highlighted that the base shear not are able to take into account the ductile capacity of the structures.

However, the Damage Model in E08 is able to bring into account the deformation capacity of structure; a better correspondence with the local limit condition (V14) is realizable considering the ultimate deformation, not the 75%.

The quantitative characterization of limit states must take into account of real capacities of structural class. In addition, if the definition of a single interstory drift value for each limit states, of a certain sub-class, is reductive because an extreme variability was been found, and the probabilistic approach is more complex, an alternative is the approach used in V14.

In fact in V14 the Damage Model has been characterized for each limit states; on the basis on the NLDAs results, an accurate assessment of repair cost is possible.

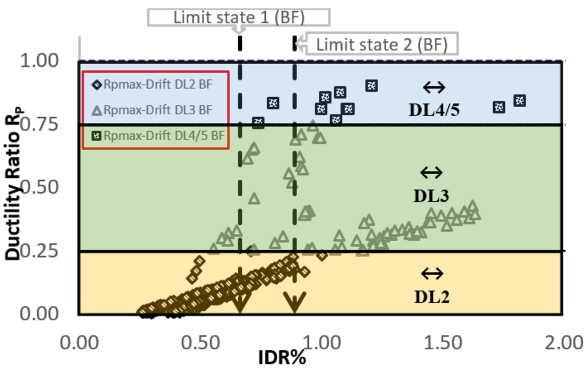

Figure 1: Comparison between the limit values IDR DL2-3-4 (V14) and LS1-LS2 (S14).

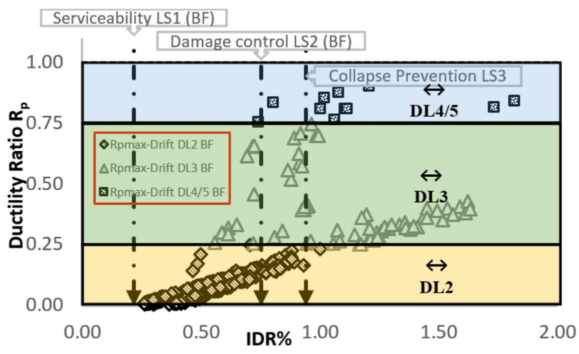

Figure 2: Comparison between the limit values IDR DL2-3-4 (V14) and LS1-LS2-LS3 (E08).

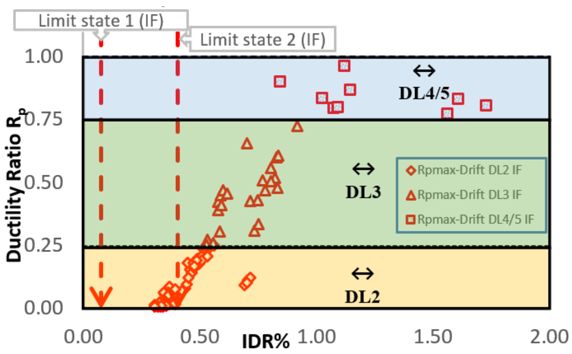

Figure 3: Comparison between the limit values IDR DL2-3-4 (V14) and LS1-LS2 (S14).

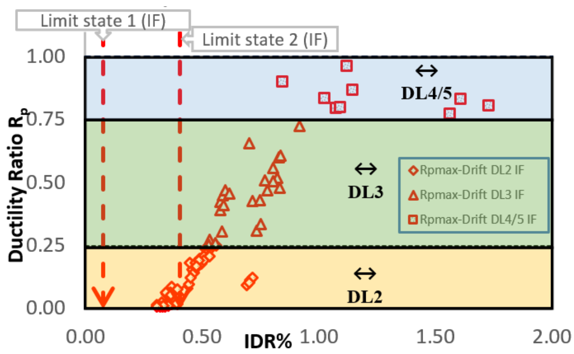

Figure 4: Comparison between the limit values IDR DL2-3-4 (V14) and LS1-LS2-LS3 (E08)

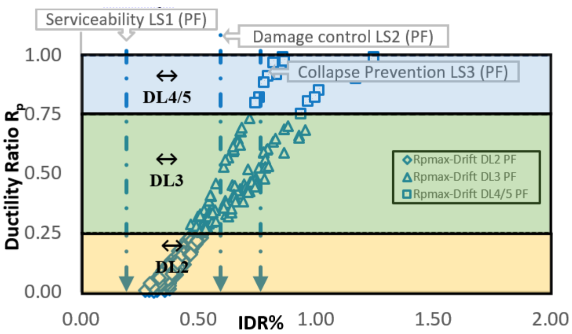

Figure 5: Comparison between the limit values IDR DL2-3-4 (V14) and LS1-LS2 (S14).

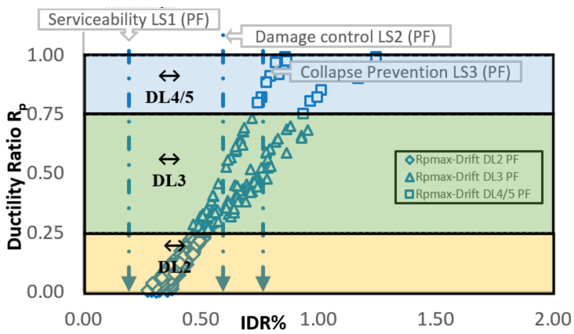

Figure 6: Comparison between the limit values IDR DL2-3-4 (V14) and LS1-LS2-LS3 (E08)

3 CONSIDERATION AND IMPROVEMENT ABOUT THE GENERATION A FRAGILITY MODEL

The accuracy of the FCs and consequent seismic risk studies (economic loss, cost-effectiveness of repairing damage and seismic retrofit) are mainly linked to structural modeling and analysis, structural performance and Damage Model. Therefore, a different efforts are need to define these topics.

The critical review of different procedures, models, choices in the construction process for fragility curves definition is carried out. A great variability in terms of geometrical, mechanical and structural characterization, structural modeling, method of analysis, scale of damage, parameters of seismic intensity and statistical procedure has been highlighted, and finally an optimal procedure of fragility analysis has been outlined.

An optimal procedure of FCs construction must be based on numerical simulations performed through NLDAs; the seismic action must been modeled by natural accelerograms. The Damage Model must been defined considering a representative limit states; they should be able to describe the different damage conditions. At each limit state must been associated a clear description of structural and nonstructural damage.

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