Evaluation of Analytical Methodologies to Derive Vulnerability Functions

V. Silva, H. Varum
University of Aveiro, Portugal

H. Crowley, R. Sousa
EUCENTRE, Pavia, Italy

R. Pinho
University of Pavia, Italy

SUMMARY
The recognition of fragility functions as a fundamental tool in seismic risk assessment has led to the development of more and more complex and elaborate procedures for their computation. Although vulnerability functions have been traditionally produced using observed damage and loss data, more recent studies propose the employment of analytical methodologies as a way to overcome the frequent lack of post-earthquake data. The variation of the structural modelling approaches on the estimation of building capacity has been the target of many studies in the past, however, its influence in the resulting vulnerability model, impact in loss estimations or propagation of the uncertainty to the seismic risk calculations has so far been the object of restricted scrutiny. Hence, in this paper, an extensive study of static and dynamic procedures for estimating the nonlinear response of buildings has been carried out in order to evaluate the impact of the chosen methodology on the resulting vulnerability and risk outputs. Moreover, the computational effort and numerical stability provided by each approach were evaluated and conclusions were obtained regarding which one offers the optimal balance between accuracy and complexity.

Keywords: fragility, vulnerability, analytical methodologies, loss assessment

1. INTRODUCTION

Fragility functions, a fundamental component in the process of assessing seismic risk, can be defined as the probability of exceeding a set of limit states, given a certain level of ground motion. Damage of buildings from past earthquakes can be used to derive these types of functions, (Rossetto and Elnashai, 2003; Rota et al., 2006), however, empirical methodologies can have some disadvantages such as the subjectivity in allocating each building in a damage state or the lack of accuracy in the determination of the ground motion that affected the region. Furthermore, there are only a few dozen places in the world where post-earthquake damage data has been collected from a number of buildings large enough to permit the development of reliable vulnerability models. To overcome this issue, analytical methodologies can be employed in which a single structure believed to be representative of a class of buildings or a set of randomly generated buildings are modelled using finite element techniques, and tested against specific loading patterns or ground acceleration time histories (see e.g. Singhal and Kiremidjian, 1996; Dumova-Jovanoska, 2000; Akkar et al., 2005; Erberik, 2008). As discussed by Rossetto and Elnashai (2005), there is not a unique methodology for the development of fragility functions and therefore, the resulting curves will be conditional on the assumptions and techniques followed in the process. These discrepancies due to the different approaches will consequently originate significant differences in the risk assessments, even when considering the exact same region, seismicity and type of structures (Strasser et al., 2008). The various analytical methodologies can be categorized in two main groups: nonlinear dynamic analysis and nonlinear static analysis, each one having its own strengths and weaknesses. The main advantage in employing non linear dynamic analysis is certainly the fact that the actual phenomena is reproduced by applying an acceleration time history at the base of the structure, leading in theory to more accurate results. However, the intrinsic complexity (e.g.: definition of damping model, post-elastic behaviour) associated with the heavy computational effort, is often impractical, thus favouring the employment of simpler methods, comprised of nonlinear static analysis (Antoniou and Pinho, 2004). In this second approach, pushover curves are computed and crossed with non-linear static procedures to estimate the maximum displacement experienced by the structure for a given ground motion record. The main drawback of this simplified methodology lays with the assumption that the structural behaviour obtained from horizontal loading is capable of replacing the one attained in the dynamic analysis.

In this paper, several analytical methodologies are used to derive fragility functions for the same structure type. A number of static procedures are investigated based on conventional and adaptive pushover analyses together with non-linear static procedures (e.g. Capacity Spectrum Method, Displacement Coefficient Method, N2 Method), using hundreds of ground motion records, to derive fragility functions for different levels of ground motion (intensity measure levels). Then, incremental dynamic analysis is used as the baseline method in this sensitivity study, to yield conclusions regarding the accuracy of each method. Each set of fragility functions is transformed into vulnerability functions (i.e. probability of loss for a given level of ground motion) by calculating the mean damage ratio (i.e. ratio of cost of repair to cost or replacement) for a number of intensity measure levels. In all methods, hundreds of 2D reinforced concrete bare frames have been simulated using a Monte Carlo approach based on the variability in the material and geometric properties of real typical Turkish buildings.

2. DESCRIPTION OF THE FRAMEWORK

For the purposes of this study, a comprehensive probabilistic framework was developed and its architecture is schematically represented in Figure 2.1.

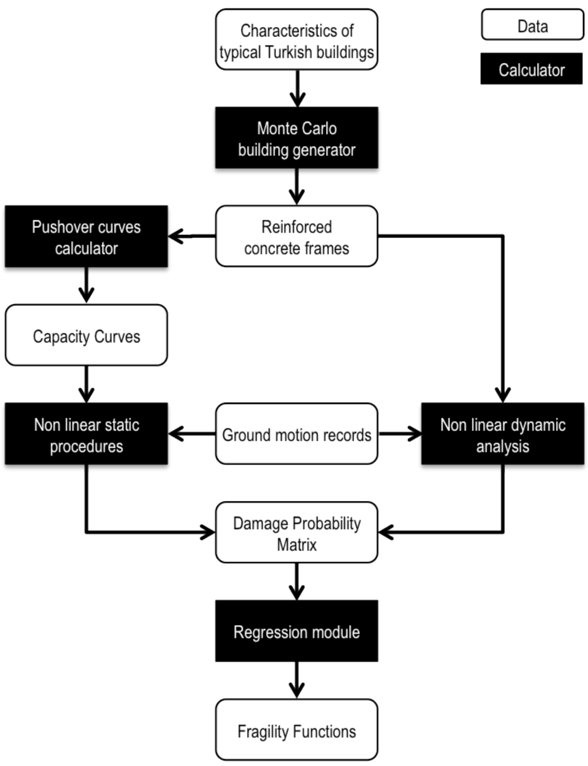

Figure 2.1. Scheme of the developed framework.

Regardless the chosen analytical methodology, the process of computing fragility curves always starts with the generation of a sample of RC structures. Then, each synthetic frame is tested against a set of ground motion records through the use of either static or dynamic analysis. At the end of this process, a distribution of buildings in each damage state for each ground motion record is obtained. As discussed in Akkar et al. (2005), there are several options regarding the criteria to allocate buildings in a damage state, such as the maximum roof displacement, interstorey drift ratio, steel or concrete strain level, maximum base shear, etc. Each option will naturally lead to different damage distributions, and consequently, different fragility functions. The influence of these criteria has been discussed in previous studies (e.g.: Priestley, 1998) and will not be further investigated in this work. In this study, the maximum top displacement has been used to identify the threshold between each damage state, as described below:

- Limit state 1: top displacement when 75% of the maximum base shear capacity is achieved;
- Limit state 2: top displacement when the maximum base shear capacity is achieved;
- Limit state 3 (or collapse): top displacement when the base shear capacity decreases 20%.

2.1 Generation of Synthetic RC frames

In all methods, hundreds of 2D reinforced concrete bare frames have been simulated using a Monte Carlo approach based on the variability in the material and geometric properties of real typical Turkish buildings. In order to maintain the computational effort at a reasonable level, a single type of frame was considered, with 4 storeys and 3 bays, as demonstrated in Figure 2.2.

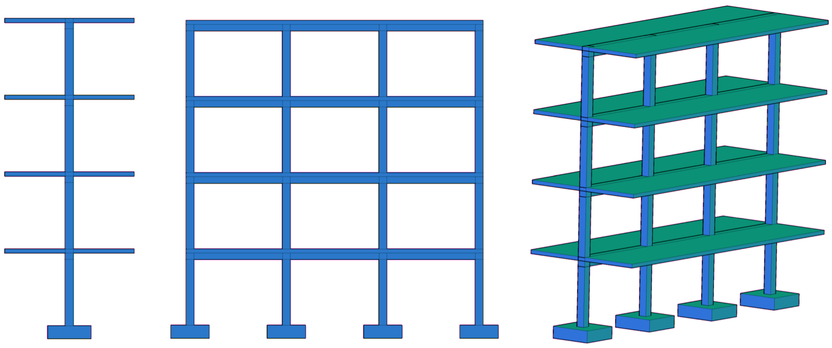

Figure 2.2. Schematic view of the RC frame model: front (left), side (centre) and isometric view (right).

A dynamic framework was developed to individually produce and design each frame. In a first phase, several parameters are randomly sampled based on the probabilistic distributions proposed by Bal et al. (2008), which are described in Table 2.2.

Table 2.2. Probabilistic distributions of the material and geometric properties.

Parameter	Mean	COV	A*	B*	Type of distribution
Steel modulus (GPa)	210	5%	-	-	Normal
Steel yield strength (MPa)	371.1	24%	-	-	Normal
Concrete yield strength (MPa)	16.7	50%	2	40	Gamma
Regular height (m)	2.84	8%	-	-	Lognormal
Ground/regular height ratio	1.13	14%	1	1.4	Exponential
Beam length (m)	3.37	38%	-	-	Gamma
Column depth (m)	0.49	30%	0.4	1	Lognormal

*A and B indicate the lower and the upper bounds respectively of the truncated distribution.

Once this set of parameters is sampled, an automatic process is triggered to calculate beam depth and area of steel of each structural element, only based on the gravity loads, as was the common practice at the time (prior to the 1999 Turkish design code). For what concerns the reinforcement steel in the columns, a limit for the area of steel of at least 1% of the concrete sectional area was set. It is important to note that Bal et al. (2008) also suggested statistics for the beam depth, but no information is provided with regards to the correlation between depth and length of the beams. Thus, independently sampling these two parameters could lead to unrealistic situations (e.g.: very long beams with a small depth). Moreover, it was assumed that each bay length would be individually sampled, but a correlation factor of 0.5 would be kept, in order to avoid the generation of highly irregular structures. Once the area of steel of the beams and columns were computed, a number of reinforcement bars capable of providing the previously estimated amount of steel were attributed to each element, completing the design of the frames. It was estimated that a minimum of 100 RC frames are required to have convergence in the results. Using a large number of specimens in vulnerability assessment, rather than just a single structure with the mean properties of a certain building class, allows the consideration of the material and geometric uncertainties in the process of deriving the vulnerability functions.

2.2 Numerical Modelling of the RC Frames

In order to use the synthetic RC frames in the various analytical analysis, the developed framework was connected to OpenSEES [1], an open source platform for structural modelling and assessment. Each frame was modelled using a 2D environment, thus considering only 3 degrees of freedom per node (2 translations, 1 rotation). The structural elements (beams and columns) were modelled using fibre sections in order to capture the non-linear behaviour of the materials. The unconfined and confined concrete were assumed to follow the Kent-Park model modified by Scott et al. (1982) with a confined coefficient equal to 1.15, whereas the behaviour of the steel was represented by the model suggested by Giuffrè and Pinto (1970). The geometric nonlinearity was also considered in the model by applying a geometric transformation of the node coordinates into the global system considering the P-delta effects. The gravity loads were applied in the structure in the form of uniform distributed loads on the beams, using a force-based approach with 25 increments.

3. PUSHOVER CURVES MODULE

3.1 Conventional Pushover

A pushover curve describes the relation between base shear and top displacement of a multi-degree of freedom (MDOF) structure when an increasing lateral force is applied. The use of pushover curves in earthquake engineering somewhat originates from the pioneering work or Gulkan and Sozen (1974), in which simplified SDOF structures were created to represent MDOF systems and used in nonlinear static analysis. Such approach has many advantages and disadvantages that have been the focus of several studies for the past years, specially the one by Krawinkler and Seneviratna (1998). The authors stated that such approach is a valuable tool in vulnerability assessment due to its simplicity, ease of use and reduced running time, despite its inability to reproduce certain phenomena such as P-delta effects, viscous damping, strength deterioration or pinching effect. The authors also highlighted the constant loading pattern as one of the weakest points of this method, as it ignores some deformation modes that are propelled by dynamic response and inelastic response characteristics. This invariant loading pattern usually adopts a uniform, triangular or a first deformation mode shape. In this study, the first two patterns were considered but not the latter since due to the high regularity of the RC frames, the first deformation mode has approximately a triangular shape, thus leading to the same structural behavior. It was decided instead, to apply a loading pattern with the resulting shape from the contribution of the first 3 modes of vibration.

The transformation of the pushover curve from the MDOF system to a capacity curve in terms of spectral acceleration (Sa) versus spectral displacement (Sd) for an equivalent SDOF structure can be carried out in various ways, always assuming that the deformed shape of the structure is not significantly altered during the dynamic loading. The top displacement was converted to Sd based on the participation factor of the first mode of vibration, while the base shear was reduced to Sa using the same factor and the first modal mass. Hundreds of capacity curves were derived for the randomly generated RC frames and the results from this module are presented in Figure 3.1, along with the mean and median capacity curves. A single RC frame was also modelled using the mean material and geometric properties (see Table 2.2), and the resulting capacity curve is also presented in the same figure.

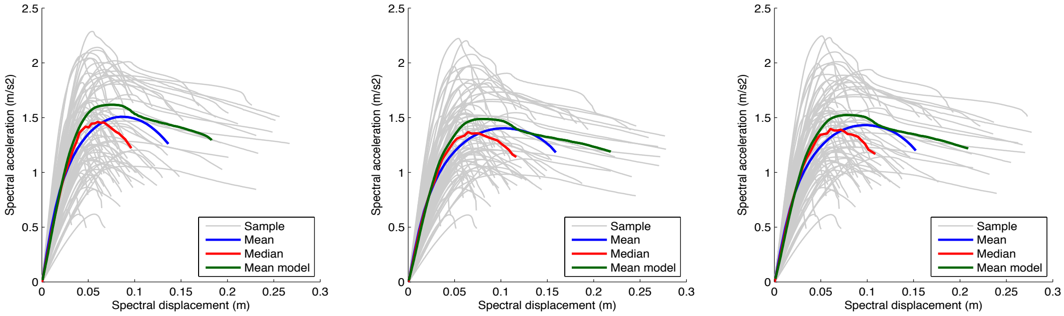

Figure 3.1. Capacity curves using a uniform (left), triangular (centre) and modal (right) loading pattern.

These results show a very large dispersion in the capacity of the RC frames, strengthening the idea that using a single or few structures to represent a building typology might be insufficient to properly capture their characteristics. Moreover, it was also noticed that the capacity curve generated using the RC frame following the mean geometric and material properties was significantly different from the mean of the capacity curves from the randomly generated frames. In fact, this capacity curve presented a considerably higher displacement capacity, which suggests that if such output would be used in seismic risk assessment, the losses could be underestimated. Regarding the differences due to the application of the different loading patterns, it was observed that applying a uniform load led to higher values of base shear capacity, whilst greater displacement capacity was attained when employing a triangular load. Such results are in agreement with other studies such as Antoniou and Pinho (2004) or Papanikolaou and Elnashai (2005). Applying a loading pattern based on the contribution of the first three modes of vibration generated intermediate results, as expected. The mean limit state spectral displacements and accelerations are presented in Table 3.2.

Table 3.2. Mean spectral displacement and acceleration for each limit state, per loading pattern.

	Uniform		Triangular		Modal	
	Sd (m)	Sa (g)	Sd (m)	Sa (g)	Sd (m)	Sa (g)
LS1	0.030	1.191	0.035	1.123	0.033	1.140
LS2	0.064	1.579	0.080	1.487	0.072	1.508
LS3	0.136	1.260	0.218	1.189	0.152	1.200

3.2 Adaptive Pushover

As an attempt to overcome some of the previously mentioned shortcomings of conventional pushover, several authors (Bracci et al., 1997; Elnashai, 2001; Antoniou and Pinho, 2004) developed adaptive or fully adaptive pushover procedures. These innovative techniques have the advantage of better accounting for degradation characteristics, influence of higher mode effects and spectral amplifications due to ground motion frequency content. In this method, instead of applying an invariant load vector, the structural properties of the model are evaluated at each step of the analysis, and the loading pattern is updated accordingly. In this way, the variation in the structural stiffness at different deformation levels, and consequently the system degradation and period elongation can be accounted for. The only apparent drawback of this methodology can be the additional computation time required to assess the structural characteristics at every step.

In this study, a displacement based adaptive pushover technique was used, in which the SRSS approach was employed in the modal combination to update the lateral load profile. Once again, several capacity curves were derived for the randomly generated RC frames, as well for the frame model with the mean characteristics. Figure 3.2 and Table 3.3 present these results.

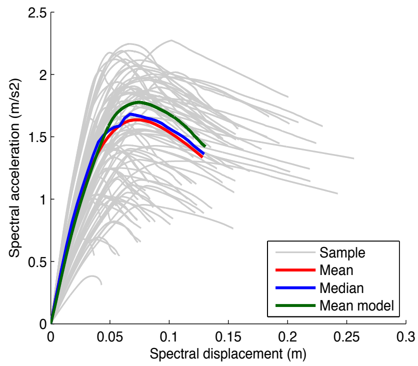

Figure 3.2. Capacity curves using DAP.

Table 3.3. Mean spectral displacement and acceleration for each limit state.

	Adaptive capacity curve	
	Sd (m)	Sa (g)
LS1	0.033	1.140
LS2	0.072	1.508
LS3	0.152	1.200

A large scatter in the capacity of the RC frames is still observed but in this case the capacity curve obtained using the model with the mean characteristics is much closer to the mean of the capacity curves. In the work of JICA (2002), Bogaziçi (2002) and Akkar et al. (2005) the lateral capacity of common building typologies in Turkey (comparable to the one that is being considered here) was evaluated and similar results with the ones attained here were observed. With regards to the variations between conventional and adaptive techniques, it is possible to conclude through the observation of median capacity curve, that the latter approach led to slightly superior base shear capacity and significantly higher top displacements. These differences in the statistics will naturally have a direct impact in the associated fragility function, as explained in the following section.

4. NONLINEAR STATIC PROCEDURES

The so-called Nonlinear Static Procedures (NSP) represent a simplified approach for the assessment of the seismic behaviour of structures. The recognition of their value boosted their application in some guidelines such as the ATC-40 (1996) and FEMA-440 (2005) in the United States or the Eurocode 8 (CEN, 2005) in Europe. In this study, three distinct methodologies were employed: the Capacity Spectrum Method (CSM) (Freeman, 1975), the Coefficient Displacement Method (CDM) (FEMA-440, 2005) and the N2 Method (Fajfar, 1999), which are further described in the following sections. These methodologies make use of capacity curves (regardless of the approach used to produce them) already after the transformation to the equivalent SDOF in terms of Sa versus Sd. Then, the Nonlinear Static Procedure is employed to estimate the target displacement for each ground motion record, and this level of displacement is used to allocate the building in a damage state. This target displacement can be equated to the maximum top displacement that would be experienced by the equivalent SDOF structure in a nonlinear dynamic analysis; this can be compared with the aforementioned limit states (see Section 2) to identify the global damage state. The distribution of buildings in each global damage state per ground motion record can then be used to derive a fragility function for each limit state, which are represented by a lognormal distribution, with a logarithmic mean (λ) and a logarithmic standard deviation (ζ). The overall process is summarized in the following manner:

1. Random generation of a population of 2D frames through Monte Carlo simulation;
2. Pushover curve for each frame, and transformation to the curve for a SDOF system;
3. Estimate target displacement for each frame, using a large selection of ground motion records and a given Nonlinear Static Procedure;
4. Identification of the global damage state based on the nonlinear response;
5. Representation of the cumulative percentage of buildings in each damage state versus the representative parameter of the each accelerogram (e.g: Sa(Ty), PGA);
6. Regression analysis to calculate the parameters (mean and standard deviation) of the fragility functions (assumed to follow a lognormal distribution)

The selection of the ground motion records was done based on the seismicity and fault rupture mechanism in the area of interest, as described in Silva et al. (2012).

4.1 Capacity Spectrum Method

The capacity spectrum method (CSM) was initially proposed by Freeman et al. (1975), and it represents a simplified methodology for many purposes such as the evaluation of a large inventory of buildings, assessment of new or existing structures or to identify the correlation between damage states and level of ground motion (Freeman, 2004). This procedure iteratively compares the capacity and the demands of a structure, using a pushover curve (for the simplified SDOF) and a response spectrum, respectively. The ground motion spectrum is computed for a level of equivalent viscous damping calculated at each iteration, in order to take into account the inelastic behaviour of the structure. The final intersection of these two curves approximates the response of the structure. The capacity curves computed in the previous sections were used with this nonlinear static procedure to derive fragility functions for each limit state, as presented in Table 4.1.

Table 4.1. Statistics of fragility functions produced using the CSM.

	Uniform		Triangular		Modal		DAP	
	λ	ζ	λ	ζ	λ	ζ	λ	ζ
LS1	-2.071	0.225	-2.121	0.261	-2.169	0.259	-2.070	0.317
LS2	-1.439	0.323	-1.364	0.254	-1.438	0.285	-1.327	0.291
LS3	-1.084	0.575	-1.043	0.508	-1.043	0.552	-0.975	0.408

4.2 Displacement Coefficient Method

The Displacement Coefficient Method (DCM) represent a methodology for the assessment of the seismic response of a building, proposed initially in ATC-40 (1996) and further developed in FEMA-440 (2005). This method consists of modifying the elastic spectral displacement for the effective fundamental period (extracted from the capacity curve), according to four coefficients. These four parameters have the purpose of introducing the effect of the difference in the response of the SDOF and the MDOF systems, the variation between elastic and inelastic response, possible degradation of stiffness and energy dissipation and the effect of P-delta effects (Lin et al., 2004). The statistics of the fragility functions obtained by crossing this nonlinear static procedure with the pushover curves produced previously are presented in Table 4.2.

Table 4.2. Statistics of fragility functions produced using the CDM.

	Uniform		Triangular		Modal		DAP	
	λ	ζ	λ	ζ	λ	ζ	λ	ζ
LS1	-2.133	0.269	-2.055	0.194	-2.126	0.296	-2.080	0.292
LS2	-1.424	0.376	-1.384	0.383	-1.394	0.376	-1.471	0.362
LS3	-0.932	0.607	-0.856	0.673	-0.882	0.660	-0.823	0.526

4.3 N2 Method

Fajfar (1999) firstly proposed this simplified nonlinear procedure for the estimation of the seismic response of structures. It is somehow similar to the Capacity Spectrum Method as it also uses capacity curves and response spectra, but it differs in the fact that it uses inelastic spectra rather than elastic spectra for an equivalent period. Moreover, it also has the distinct aspect of assuming an elasto-perfectly plastic force-displacement relationship. In order to estimate the target displacement, it is necessary to assess whether the SDOF structure is in the short-period or medium and long-period range. Then, if the structure is in the latter category, it is assumed that the target displacement is equal to the elastic spectral displacement for the fundamental period. If on the other hand it is located in the short-period range, a simple procedure is carried out to understand if the response is going to be elastic or inelastic, and in the second case, a formula that takes into account the nonlinear behaviour of the structure is applied. The statistics of the fragility functions computed using this method and the previously produced capacity curves are presented in Table 4.3.

Table 4.3. Statistics of fragility curves produced using the N2 Method.

	Uniform		Triangular		Modal		DAP	
	λ	ζ	λ	ζ	λ	ζ	λ	ζ
LS1	-2.133	0.313	-2.047	0.193	-2.061	0.235	-2.112	0.292
LS2	-1.477	0.359	-1.432	0.354	-1.414	0.353	-1.440	0.346
LS3	-0.941	0.627	-0.864	0.676	-0.878	0.661	-0.813	0.536

5. NONLINEAR DYNAMIC ANALYSIS

Nonlinear dynamic analysis has been accepted as the most accurate and reliable methodology to estimate the seismic response of structures. However, design practitioners are still struggling with this subject, due to the fact that it requires advanced knowledge in structural dynamic and inelastic behaviour (Elnashai, 2001). The requirements around this approach in comparison to the previously presented nonlinear static procedures are considerably more demanding, mainly on the level of detail of the model, the necessity to represent the masses in the structure, the need to model the damping, the definition of time integration algorithms and the treatment of the ground motion input. This higher level of complexity means a significant increase in the computing time. In this study, nonlinear time history analyses were performed for several randomly generated frames, against a set of ground motion records. These accelerograms were filtered and trimmed based on the 5% of maximum PGA threshold, as described in Bommer and Pereira (1999). For each record, the percentage of RC frames in each damage state was estimated and the associated fragility functions are presented Figure 5.1 and their statistics described in Table 5.1.

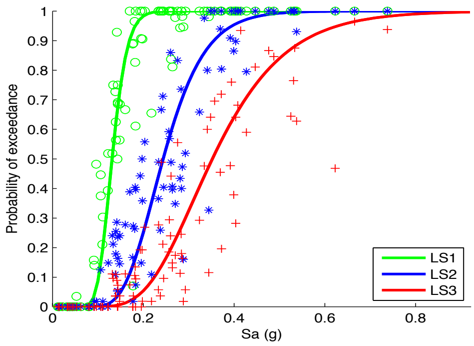

Figure 5.1. Fragility Functions using nonlinear dynamic analysis.

Table 5.1. Statistics of the fragility functions using nonlinear dynamic analysis.

	λ	ζ
LS1	-2.098	0.298
LS2	-1.448	0.354
LS3	-0.809	0.514

6. DISCUSSION OF THE RESULTS

With regards to the variation of the capacity curves based on the method of calculation, it was observed consistently an underestimation of the capacity of the randomly generated RC frames when employing conventional pushover procedures, in comparison with the adaptive pushover technique. This behaviour is due to the fact that in the former approach, the structures are forced to deform in an unnatural manner. In order to evaluate the impact that such variations would have in loss assessment, the fragility curves produced according to each combination were crossed with consequence functions (i.e.: ratios of cost of repair to cost of replacement per damage state) to derive vulnerability functions (loss ratio versus intensity measure levels). The results per pushover technique, for each nonlinear static procedure are presented in Figure 6.1.

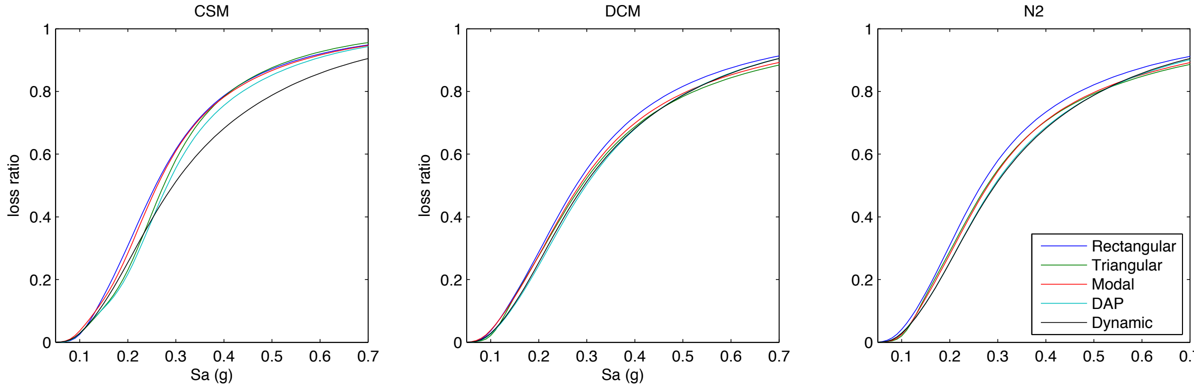

Figure 6.1. Vulnerability functions per pushover technique, for each NSP.

As expected, regardless the nonlinear static procedure, the employment of adaptive pushover techniques leads to lower loss ratios in the vulnerability. The vulnerability curves in which the Capacity Spectrum Method was employed presented consistently higher loss ratios. With regards to the other two nonlinear static procedures, the results are very similar, and usually lower than the ones obtained using the CSM. Considering all of the results, the more conservative vulnerability function is obtained when employing the N2 method + adaptive pushover curves, whilst the highest loss ratios are originated when using the Capacity Spectrum Method combined with uniform-based pushover curves. In fact, a mean and maximum average difference of 14% and 21% respectively are observed. This underestimation in the capacity of the structures when employing the CSM was also verified in an experimental exercise performed by Lin et al. (2004), in which the estimated seismic response was 20% lower than what was experimentally observed.
Finally, with regards to the differences between the results obtained using the aforementioned combinations and the ones attained with the nonlinear dynamic analysis, it was observed that the N2 method associated with the adaptive capacity curves led to almost identical results.

7. CONCLUSIONS

In this study, 12 sets of fragility functions were produced based on many different combinations of pushover curves and nonlinear static procedures, as well as a set of fragility functions using a nonlinear dynamic approach. Despite the differences in the methodologies, the comparison of the vulnerability curves obtained from each combination do not show significant discrepancies in the distributions of loss ratios.

Considering the results from the nonlinear dynamic analysis as the baseline method, it is fair to state that the application of the DCM or the N2 method gave more accurate results than those provided by the CSM. Although very different from a practical point of view, the DCM and the N2 method both use inelastic spectra to estimate the target displacement, rather than the equivalent damping approach used by the CSM, which is probably the cause of the similarity in the results. Furthermore, the employment of N2 combined with adaptive pushover curves provided results very close to those attained with the nonlinear dynamic analysis, which shows that a simplified methodology with a much lower computational effort, can still provide reliable and accurate results.

REFERENCES

Akkar, S., Sucuoglu, H., and Yakut, A. (2005) "Displacement-based fragility functions for low- and mid-rise ordinary concrete buildings", *Earthquake Spectra* 21(4), 901-927.

Antoniou, S. and Pinho, R. (2004). "Development and verification of a displacement-based adaptive pushover procedure". *Journal of Earthquake Engineering*, 8(5), pp. 643-661

ATC-40 (1996). Seismic Evaluation and Retrofit of Concrete Buildings, Volumes 1 and 2, Report No. ATC-40, Applied Technology Council, Redwood City, CA

Bal, I.E., Crowley, H., Pinho, R. and Gulay, F.G. (2008) "Detailed assessment of structural characteristics of Turkish RC building stock for loss assessment models," *Soil Dynamics and Earthquake Engineering*, Vol.

28, pp. 914–932.
Bogaziçi (2002). “Earthquake Risk Assessment for Istanbul Metropolitan Area”, Final Report, Kandilli Observatory and Earthquake Research Center, Istanbul.
Bommer, J. J. and Martínez-Pereira, A. (1999). “The effective duration of earthquake strong motion”. *Journal of Earthquake Engineering*, 3(2), pp. 127-172
Bracci, J. M., Kunnath, S. K. and Reinhorn, A. M. (1997). “Seismic Performance and Retrofit Evaluation of Reinforced Concrete Structures”. *Journal of Structural Engineering*, 123(1), pp. 3-10.
CEN (2005). Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General rules, seismic actions and rules for buildings. EN 1998-2, Comité Européen de Normalisation, Brussels, Belgium.
Dumova-Jovanoska, E. (2004). “Fragility Curves for RC Structures in Skopje Region”, *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, Canada, Paper No. 3 (on CD).
Elnashai, A. S. (2001). “Advanced inelastic static (pushover) analysis for earthquake applications”. *Structural Engineering and Mechanics*, 12(1), pp. 51-69
Erberik, M., A. (2008) “Fragility-based assessment of typical mid-rise and low-rise RC buildings in Turkey” *Engineering Structures* 30(5), 1360-1374.
FEMA-440 (2005). Impovement of Nonlinear Static Seismic Analysis Procedures, Report No. FEMA-440, Federal Emergency Management Agency, Washington, DC.
Freeman, S. A., Nicoletti, J. P. and Tyrell, J. V. (1975). “Evaluation of Existing Buildings for seismic risk - A case study of Puget Sound Naval Shipyard, Bremerton, Washington”. *Proceedings of the 1st U.S. National Conference on Earthquake Engineering*, Berkley, USA.
Giuffrè, A. and Pinto, P.E. (1970) “Il comportamento del cemento armato per sollecitazioni cicliché di forte intensità”, *Giornale del Génio Civile* (in Italian).
Gulkan, P. and Sozen, M. A. (1974). “Inelastic Responses of Reinforced Concrete Structure to Earthquake Motions”. *ACI Journal Proceedings*, 71(12), pp. 604-610.
JICA (2002). “The Study on a Disaster Prevention/Mitigation Basic Plan in Istanbul Including Seismic Microzonation in the Republic of Turkey”, *Final Report*, Istanbul.
Kalkan, E., Gülkan, P., Yilmaz, N., Celebi, M. (2008) “Probabilistic Seismic Hazard Mapping of the Marmara Region”, Report No. 08-01, Earthquake Engineering Research Center.
Krawinkler, H. and Seneviratna, G. (1998). “Pros and cons of a pushover analysis of seismic performance evaluation”. *Engineering Structures*, 20(4-6), pp. 452-464.
Papanikolaou, V. and Elnashai, A. (2005). “Evaluation of Conventional and Adaptive Pushover Analysis I: Methodology”. *Journal of Earthquake Engineering*, 9(6), pp. 923-941.
Priestley, M. (1998). “Displacement-based approaches to rational limit states design of new structures”. *Proceedings of 11th European Conference on Earthquake Engineering*, Rotterdam, The Netherlands.
Rossetto, T. and Elnashai, A. (2003). “Derivation of vulnerability functions for European-type RC structures based on observational data”. *Engineering Structures* 25(10), 1241-1263.
Rossetto, T. and Elnashai, A. (2005) “A new analytical procedure for the derivation of displacement-based vulnerability curves for populations of RC structures”. *Engineering Structures* 27(3), 397-409.
Rota, M., Penna, A. and Strobbia, C. (2008) “*Processing Italian damage data to derive typological fragility curves*”. Soil Dynamics and Earthquake Engineering 28(10-11), 933-947.
Scott, B. D., Park, R. and Priestley, M. J. N. (1982). “Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates”. *ACI Journal Proceedings*, 79(1), pp. 13-27.
Shome, N., Cornell, C. A., Bazzurro, P., and Carballo, J. E., 1998. Earthquakes, records and nonlinear responses, *Earthquake Spectra* 14, 469-500.
Silva, V., Crowley, H., Varum, H., Pinho, R. (2012) “Extending Displacement-Based Earthquake Loss Assessment (DBELA) for the Computation of Fragility Curves,” Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal.
Singhal A, Kiremidjian, S. (1997) “A method for earthquake motion–damage relationships with application to reinforced concrete frames”. NCEER report, NCEER-97-0008, State University of New York at Buffalo, USA.
Strasser, F. O., Bommer, J. J., Sesetyan, K., Erdik, M., Cagnan, Z., Irizarry, J., Goula, X., Lucantoni, A., Sabetta, F., Bal, I. E., Crowley, H., and Lindholm, C. (2008). “A comparative study of European earthquake loss estimation tools for an earthquake scenario in Istanbul”. *Journal of Earthquake Engineering*, 12(S2), pp.246–256.

Web References
[1] OpenSEES: http://opensees.berkeley.edu/