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Derivation of fragility curves using inelastic time-history analysis and damage statistics

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Summary

A methodology is presented for the derivation of seismic vulnerability (fragility) curves based on the hybrid approach, which combines statistical data with appropriately post-processed (on the basis of repair cost models) nonlinear dynamic analyses that permit extrapolation of statistical data to earthquake intensities for which no data are available. An extensive numerical study is carried out, wherein a large number of building types (representing most of the typologies common in S. Europe) are modelled and subjected to a set of ground motions. Fragility curves expressed as cumulative lognormal distributions are then derived using the aforementioned hybrid approach; the curves correlate peak ground acceleration (PGA) to the probability that a building type exceeds a particular damage state.

Introduction

As documented in the literature, seismic fragility curves can be described by (cumulative) normal, lognormal, beta or other distribution, provided that sufficient data is available for constructing them. This data might come from damage statistics from past earthquakes [1, 12], analysis of appropriate mechanical models [3, 11], or expert judgement [2]. The most common problem when applying a purely empirical approach is the unavailability of (reliable) statistical data for several intensities. By definition, intensities up to 5 lead to negligible damage, particularly cost-wise, therefore gathering of damage data is not feasible, while on the other hand events with intensities greater than 9 are rare, especially in Europe, so there are not enough data available. This unavailability leads to a relative abundance of statistical data in the intensity range from 6 to 8 and a lack of data in the other intensities, making the selection of an appropriate cumulative distribution very unreliable, since the curve fit error is significant and the curve shape not as expected. On the other hand, purely analytical approaches should be avoided, since they might seriously diverge from reality, typically (but not consistently) overestimating the cost of damage. Finally, the ATC-13 fragility curves based on expert judgement were found to grossly over-predict structural damage, at least for some classes of structures for which damage statistics were compiled [1].

In order to overcome these problems, the first author and his co-workers [5, 8] have developed the so-called ‘hybrid’ approach, a method that starts from available damage statistics (appropriate for the area and structural typology under consideration) and estimates damage at the intensities for which no data is available using analytical (nonlinear) simulation. Previous versions of the method involved multi-linear fragility curves, wherein the level of ground motion was expressed in terms of macroseismic intensity, and points corresponding to I=6 to 9 were estimated from the hybrid approach, while the remaining points (I>9) were adopted from ATC-13; these curves covered a number of building typologies common in S. Europe and were used in some risk assessment studies, e.g. of the greek city of Volos [7], the first study of this kind in Greece. In the present paper, an improved version of the method is developed, based on

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a more rigorous approach for deriving fragility curves expressed as lognormal cumulative distributions; these curves are derived in terms of PGA rather than intensity, and cover the entire range of feasible PGA's, as well as all common reinforced concrete (R/C) building types.

Building types analysed

Using the procedures described in the following, analysis of several different R/C building configurations has been performed. Referring to the height of the buildings, 2-storey, 4-storey, and 9-storey R/C buildings were analysed. Regarding the structural system, both frames and dual (frame+shear wall) systems were addressed. Each of the above buildings was assumed to have three different configurations, namely bare, infilled and pilotis (soft ground storey) type. Two seismic code levels were considered: low (early seismic codes) and high (modern seismic codes); the specific codes applied for designing the structures were the 1959 and the 2000 Greek Codes (the latter is similar to the 1995 code). To keep the cost of analysis within reasonable limits, all buildings were analysed as 2D structures. Typical structures studied are shown in Fig. 1 and 2.

The nomenclature used for the buildings is of the type RC*ij* where *i* indicates the structural system and takes the values 1 (bare frame), 3.1 (infilled frame), 3.2 (pilotis frame), 4 (bare dual system), 4.1(infilled dual), 4.2 (dual with pilotis), while *j* takes the values L(ow-rise), M(edium-rise) or H(igh-rise). Note that two more typologies, RC2 (wall systems) and RC5 (precast systems), are found in some countries, but were not considered herein. In total 36 structures were analysed in the present study.

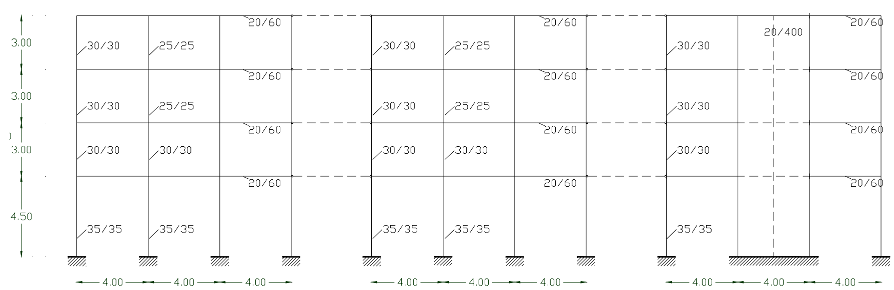

Fig. 1 4-storey building with dual system (bare structure)

Procedure for the construction of fragility curves

The vulnerability (fragility) curves are presented in the following in terms of PGA; it is recalled herein that as long as a certain empirical (attenuation) relationship between I and PGA is adopted, the two forms of fragility curves (in terms of I or PGA) are exactly equivalent. The assignment of a PGA to the statistical damage database [10] used within the hybrid method was made using the relationship *ln(PGA)=0.74*I+0.03* which is the most recent one proposed for Greece [9] and is based on statistical processing of a large number of Greek strong ground motions; this equation is calibrated for intensities less than 9, and should not be used for I>9.

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Assuming a lognormal distribution (common assumption in seismic fragility studies, e.g. [1]), the conditional probability of being or exceeding, a particular damage state ds_i, given the peak ground acceleration (PGA) is defined by the relationship

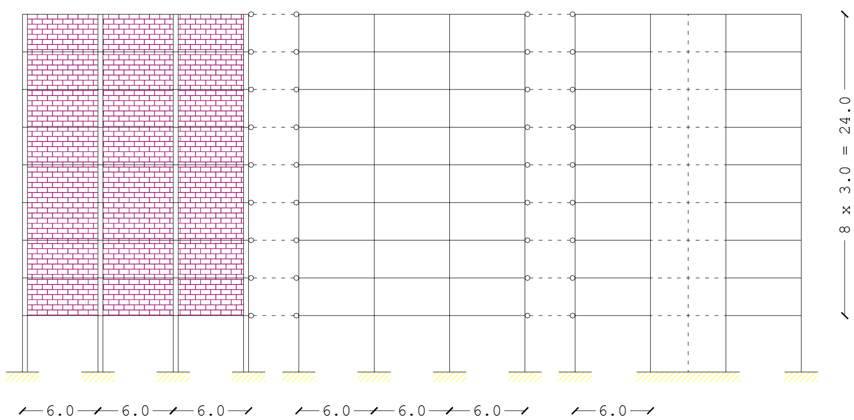

Fig. 2 9-storey building with dual system and pilotis

$$P[ds \geq ds_i/PGA]=\Phi[\frac{1}{\beta_{ds_i}}\ln(\frac{PGA}{\overline{PGA},ds_i})],$$ where

$\overline{PGA},ds_i$ is the median value of peak ground acceleration at which the building reaches the threshold of damage state, ds_i, see also Table 1 in next section

β_{ds_i} is the standard deviation of the natural logarithm of peak ground acceleration for damage state, ds_i, and

Φ is the standard normal cumulative distribution function.

Each fragility curve is defined by a median value of peak ground acceleration that corresponds to the threshold of that damage state and by the variability associated with that damage state; these two quantities are derived as described in the following. Median values for each damage state in the fragility curves were estimated for each of the 36 types of building systems considered. These values are produced based on the hybrid approach, which combines inelastic dynamic analysis and the aforementioned database of the Thessaloniki earthquake of 1978, corresponding to an intensity I=6.5, to which a peak ground acceleration of 0.13g corresponds, according to the adopted Intensity-PGA relationship; it is noted that this PGA practically coincides with the one of the only record available from the 1978 earthquake in Thessaloniki. From the database of the Thessaloniki earthquake the damage index, defined as the ratio L of repair cost to replacement cost, corresponding to this PGA is found for each building (a total of 5700 R/C buildings are included in the database).

Using the DRAIN2000 code [6], inelastic dynamic analyses are carried out for each building type and for several PGA values, until 'failure' is detected. R/C members were modelled

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using lumped plasticity beam-column elements, while infill walls were modelled using shear panel isoparametric elements developed in previous studies [6, 8]. A total of 16 accelerograms were used (to account for differences in the spectral characteristics of the ground motion), scaled to each PGA value, hence resulting to several thousands of inelastic time-history analyses. From each analysis, the cost of repair (which is less than or equal to the replacement cost) is estimated for the building type analysed, using the models for member damage indices proposed by Kappos et al. [8]. The total loss for the entire building is derived from empirical equations (calibrated against cost of damage data from Greece)

$$L = 0.25D_c + 0.08D_p \qquad (\leq 5 \text{ storeys})$$

$$L = 0.30\ D_c + 0.08\ D_p \qquad (6 - 10 \text{ storeys})$$

where D_c and D_p are the global damage indices (≤1) for the R/C members and the masonry infills of the building, respectively. Due to the fact that the cost of the R/C structural system and the infills totals less than 40% of the cost of a (new) building, the above relationships give values up to 38% for the loss index L, wherein replacement cost refers to the entire building. In the absence of a more exact model, situations leading to the need for replacement (rather than repair/strengthening) of the building are identified using failure criteria for members and/or storeys, as follows:

• In R/C *frame* structures (RC1 and RC3 typology), failure is assumed to occur (and then L=1) whenever *either* 50% or more of the columns in a storey 'fail' (i.e. their plastic rotation capacity is less than the corresponding demand calculated from the inelastic analysis), *or* the interstorey drift exceeds a value of 4% at any storey (see also [3]).

• In R/C *dual* structures (RC4 typology), failure is assumed to occur (and then L=1) whenever *either* 50% or more of the columns in a storey 'fail', *or* the walls (which carry most of the lateral load) in a storey fail, *or* the interstorey drift exceeds a value of 2% at any storey (drifts at failure are substantially lower in systems with R/C walls).

This is a new, more refined, set of failure criteria (compared to those used in previous studies by the authors, e.g. [7, 8]) and they resulted after carrying out a large number of (time-history) analyses. Although they represent the group's best judgement (for an analysis of the type considered herein), it must be kept in mind that situations close to failure are particularly difficult to model, and all available procedures have their own limitations. The present analysis (as well as most other published procedures) do not directly model failure of vertical members (beam failure *is* modelled, see [3]), hence the criterion of 50% or more 'failures' of columns in a storey is just an assumption trying to strike a balance between the (usual) overconservative approach, according to which failure coincides with failure of the first column (or wall) and the reality witnessed after earthquakes that buildings (even old ones) still stand (i.e. do not collapse) although a number of their vertical members have failed badly. As a result of the above assumptions, although in most cases the earthquake intensity estimated to correspond to failure (DS5) is of a reasonable magnitude, in some cases (in particular wall/dual structures, especially if designed to modern codes) PGA's associated with failure (see Tables 2, 3) are clearly unrealistic and should be revised in the near future. Having said this, their influence in a risk analysis is typically limited, since the scenario earthquakes do not lead to accelerations more than about 1g (recorded peak accelerations in strong events are also of this order). Having established the loss index L, the empirical cost data ("actual" cost, C_{act}) available in the database is scaled by the ratio of analytically predicted cost of damage (C_{anal}=L .V, where V

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scaled by the ratio of analytically predicted cost of damage (C_{anal}=L .V, where V the replacement value) for any two intensities under consideration. It is noted that the ratios C_{act}/C_{anal} calculated for the Thessaloniki 1978 data were reasonably close to 1.0 when the entire building stock was considered, but discrepancies for some individual building classes did exist [8]. In this way it is possible to establish a relationship between damage index and PGA for each building type, such as the one shown in Fig. 3, and consequently to assign a median value of PGA to each damage state (described by its central damage factor, CDF, see Table 1).

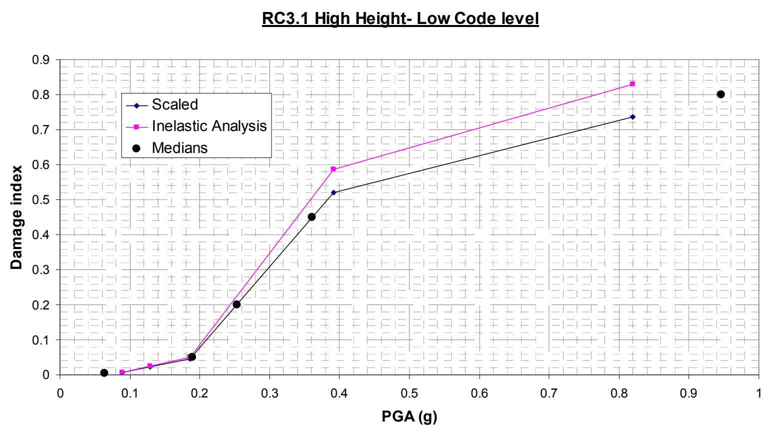

Fig. 3 PGA-Damage index relation

Lognormal standard deviation values (β) describe the total variability associated with each fragility curve. Three primary sources contribute to the total variability for any given damage state [4], namely the variability associated with the discrete threshold of each damage state which is defined using damage indices (in the present study this variability includes also the uncertainty in the models correlating structural damage indices to *loss*, i.e. the ratio of repair cost to replacement cost, see also [5]), the variability associated with the capacity of each structural type, and finally the variability of the earthquake ground motion. The uncertainty in the *definition of damage state*, for all building types and all damage states, was assumed to be β=0.4 [4], the variability of the *capacity* for low code buildings is assumed to be β=0.3 and for high code β=0.25 [4], while the last source of uncertainty, associated with *seismic demand*, is taken into consideration through a convolution procedure, i.e. by calculating the variability in the final results of inelastic dynamic analyses carried out for a total of 16 motions at each level of PGA considered.

Results of analysis

Table 1 provides the best estimate values for the damage index ranges associated with each damage state, derived from previous experience of the authors in R/C structures. The parameters of the cumulative normal distribution functions derived for all R/C structures designed to 'low-code' are given in Table 2; similar results are available for all other cases studied. Example fragility curves constructed are given in Figures 5 and 6. Referring first to Table 2, it is noted that median values are expressed as fractions of g, and that beta-values are con-

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stant for each building type; this constant value is the average of the 5 values of beta corresponding to each of the 5 damage states. This was done on purpose, because if the (generally) different variability associated with each damage state (calculated from the results of time-history analysis) is taken, unrealistic fragility curves (for instance, intersecting) result in cases that median values are closely spaced (e.g. see Fig. 5-left). As can be seen in Fig. 5, the effect of seismic design is significant in the case of infilled frames; much less influence of the seismic design level was found regarding the vulnerability of dual buildings (Fig. 6), an observation which is consistent with observed behaviour of old R/C buildings (i.e. buildings with walls perform consistently better than those with frames only).

Table 1 Damage grading and loss indices

Damage State	Damage state label	Range of damage state	Central damage state (%)
DS0	None	0.0	0.0
DS1	Slight	0.-1.	0.5
DS2	Moderate	1.-10.	5.0
DS3	Substantial to heavy	10.-30.	20.
DS4	Very heavy	30.-60.	45.
DS5	Destruction	60.-100.	80.

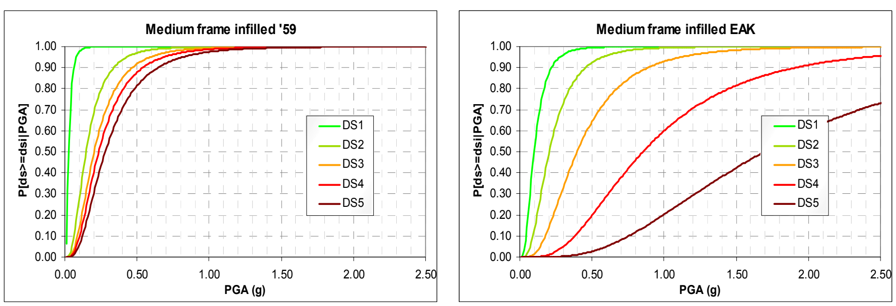

Fig. 5 Fragility curves for medium-rise infilled frames, low (left) and high code design.

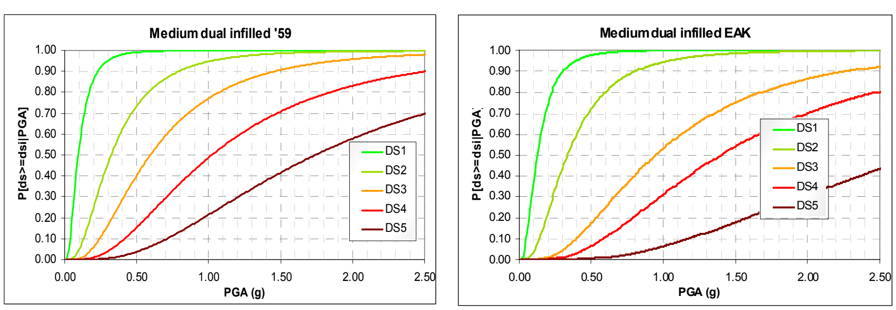

Fig. 6 Fragility curves for medium-rise dual infilled systems, low (left) and high code design.

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Table 2 Estimated fragility parameters for low code design

BTM	Slight		Moderate		Substantial to heavy		Very Heavy		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta	Median	Beta
RC1L	0.006	0.707	0.058	0.707	0.127	0.707	0.195	0.707	0.251	0.707
RC1M	0.007	0.707	0.065	0.707	0.116	0.707	0.166	0.707	0.216	0.707
RC1H	0.030	0.707	0.114	0.707	0.215	0.707	0.367	0.707	0.825	0.707
RC3.1L	0.091	0.707	0.184	0.707	0.229	0.707	0.300	0.707	0.413	0.707
RC3.1M	0.027	0.707	0.146	0.707	0.203	0.707	0.235	0.707	0.280	0.707
RC3.1H	0.064	0.707	0.189	0.707	0.253	0.707	0.360	0.707	0.946	0.707
RC3.2L	0.024	0.707	0.099	0.707	0.148	0.707	0.207	0.707	0.261	0.707
RC3.2M	0.002	0.707	0.021	0.707	0.083	0.707	0.118	0.707	0.160	0.707
RC3.2H	0.093	0.707	0.159	0.707	0.281	0.707	0.502	0.707	0.901	0.707
RC4L	0.026	0.707	0.158	0.707	0.277	0.707	0.453	0.707	0.730	0.707
RC4M	0.016	0.707	0.119	0.707	0.304	0.707	0.580	0.707	0.954	0.707
RC4H	0.009	0.707	0.097	0.707	0.331	0.707	1.375	0.707	3.099	0.707
RC4.1L	0.095	0.707	0.244	0.707	0.458	0.707	0.627	0.707	0.865	0.707
RC4.1M	0.094	0.707	0.322	0.707	0.594	0.707	0.967	0.707	1.488	0.707
RC4.1H	0.097	0.707	0.206	0.707	0.381	0.707	1.584	0.707	3.312	0.707
RC4.2L	0.070	0.707	0.280	0.707	0.464	0.707	0.617	0.707	0.832	0.707
RC4.2M	0.091	0.707	0.237	0.707	0.442	0.707	0.673	0.707	0.995	0.707
RC4.2H	0.100	0.707	0.214	0.707	0.516	0.707	1.518	0.707	2.922	0.707

Conclusions

A feasible procedure for constructing fragility curves using the "hybrid" technique, which combines statistical data with appropriately post-processed results of nonlinear dynamic analyses, was described in this paper. This procedure has been applied to derive fragility curves for a total of 36 R/C building types, covering most of the common typologies in S. Europe, notably including buildings infilled with brick masonry walls, in addition to bare structures. The structures were subjected to a total of 16 ground motions scaled to different levels of PGA and the calculated damage (loss) indices were adjusted based on statistical data of seismic damage. The buildings were designed to 'low' and 'high' seismic code provisions, and the effect of seismic design was found (from the corresponding fragility curves) to be significant in the case of infilled frames; much less influence of the seismic design level was found regarding the

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vulnerability of dual buildings, an observation that is consistent with observed behaviour of old R/C buildings (structures with walls perform better).

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