Fragility curves for reinforced concrete buildings in Greece
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The latest developments of a methodology developed by the authors and their co-workers for estimating direct losses from earthquakes in reinforced concrete (R/C) buildings are presented; they concern the derivation of capacity curves and vulnerability (fragility) curves in terms of peak ground acceleration (PGA), as well as spectral displacement, for all types of R/C buildings that are common in Greece. The vulnerability assessment methodology is based on the hybrid approach, which combines statistical data with appropriately processed results from nonlinear dynamic or static analyses that permit interpolation and (under certain conditions) extrapolation of statistical data to PGAs and/or spectral displacements for which no data is available. A detailed discussion of the limitations of the hybrid approach is provided, along with a proposal for improving the quality of results by applying a weighting technique to both the analytical and the statistical input data.

Keywords: capacity curves; fragility curves; hybrid methodology; loss assessment; reinforced concrete buildings; vulnerability

1. Introduction
The last decade or so has witnessed a growing interest in assessing the seismic vulnerability of European cities and the associated risk; not surprisingly, this interest was stronger in southern Europe where the largest part of the seismic energy dissipation in this continent takes place. A fair number of earthquake damage (and loss) scenario studies appeared wherein some of the most advanced techniques have been applied to the urban habitat of European cities (Bard et al. 1995, Barbat et al. 1996, D’Ayala et al. 1996, Faccioli et al. 1999, Kappos et al. 2002, Erdik et al. 2003, Dolce et al. 2006). By ‘scenario’ it is understood here that the study refers to a given earthquake (maximum credible, or standard design, or frequent) and provides a comprehensive description of what happens when such an earthquake occurs; this is not the same as ‘risk analysis’, which refers to all possible earthquakes, estimating the probability of losses over a specified period of time. A key feature of the most recent studies is the use of advanced geographic information system (GIS) tools that permit clear representations of the expected distribution of damage in the studied area and visualisation of the effects of any risk mitigation strategy that can be adopted on the basis of the scenario.

As noted by Dolce et al. (2006), preparing a scenario requires contributions from a wide range of topics and disciplines, spanning from seismology and geology to structural and geotechnical engineering, from urban planning and transport engineering to social and economic sciences. However, it often happens that each specialist interacts very little (if at all) with other specialists. Furthermore, some key parameters that are particularly relevant for a scenario, such as the geological features that affect seismic hazard, the characteristics of the building stock and socio-economic conditions, are different in each country. This makes the common practice of adopting the same models for developing scenarios in different countries at best questionable. A good example of the problems involved in adopting models from another country is outlined in the paper by Barbat et al. (1996) who had to adapt vulnerability models developed for Italian masonry buildings to study buildings in Barcelona.

This paper focuses on the derivation of vulnerability (fragility) curves in terms of peak ground acceleration (PGA), as well as spectral displacement ($S_d$), and also includes the estimation of capacity curves, for several reinforced concrete (R/C) building types that are common in Greece, as well as the rest of Southern Europe. The vulnerability assessment...
methodology is based on the hybrid approach developed at the Aristotle University of Thessaloniki (AUTh) by the authors and their co-workers (Kappos et al. 1998b, 2002, 2006); this approach combines statistical data with appropriately processed results (utilising repair-cost models) from nonlinear dynamic or static analyses, which permit interpolation and (under certain conditions) extrapolation of statistical data to PGAs and/or spectral displacements for which no data are available. The statistical data used herein are from earthquake-damaged Greek buildings.

An extensive numerical study was carried out, wherein a large number of building types (54 in total), representing most of the common typologies in Southern Europe, were modelled and analysed. Buildings were defined on the basis of material, structural system, height and age (which indirectly defines also the code used for design, if any), and the existence or otherwise of brick masonry infills. The RC building models were analysed for a total of 16 carefully selected accelerograms (half of them from actual Greek records, the other half synthetic) that are representative of different ground conditions. Vulnerability curves for several damage states were then derived using the aforementioned hybrid approach. These curves were subsequently used, in combination with appropriately defined response spectra, for the derivation of new vulnerability curves involving spectral quantities. Pushover curves were derived for all building types (RC), then reduced to standard capacity curves, and can be used together with the $S_d$-based fragility curves as an alternative for developing seismic risk scenarios.

Although the hybrid approach is, at least in our opinion, the best available for deriving fragility curves, it is not problem-free, in the sense that extrapolation (and sometimes even interpolation) of damage data is questionable, given the nonlinear nature of the relationship between earthquake intensity and level of structural damage (Kappos 1997). Hence, a detailed discussion of the limitations of the hybrid approach is provided herein, along with a proposal for improving the quality of results by applying a weighting technique to both the analytical and the statistical input data. It is worth pointing out that the way fragility curves were developed here using the hybrid approach at the stage of producing damage degree versus earthquake intensity relationships is different to other procedures in the literature, which are based either on fitting of curves directly to empirical data (e.g. Spence et al. 1992), or on expert judgement (e.g. ATC 1985), or on separate empirical and analytical procedures without combining the two (Lagomarsino and Giovinazzi 2006).

2. Buildings analysed

Using the procedures described in the following, analysis of several different R/C building configurations has been performed, representing practically all common R/C building types in Greece and many other Southern European countries. Referring to the height of the buildings, two-, four- and nine-storey R/C buildings were selected as representative of low-, medium- and high-rise buildings, respectively. The nomenclature used for the buildings is of the type RC$\text{ixy}$, where $i$ indicates the structural system, $x$ the height and $y$ the code level. Regarding the structural system, both frames (RC1 and RC3 types) and dual (frame + shear wall, RC4) systems were addressed. Each of the above buildings was assumed to have three different configurations: ‘bare’ (without masonry infill walls, RC1 type), ‘regularly infilled’ (RC3.1) and ‘irregularly infilled’ (soft ground storey, usually pilotis, RC3.2 type).

Regarding the level of seismic design and detailing, four subclasses could be defined, as follows:

- No code (or pre-code): R/C buildings with a very low level of seismic design or no seismic design at all, and poor quality of detailing of critical elements, e.g. RC1MN (medium rise, no code).
- Low code: R/C buildings with a low level of seismic design (roughly corresponding to pre-1980 codes in Southern Europe, such as the 1959 code for Greece), e.g. RC3.2LL (low rise, low code).
- Moderate code: R/C buildings with a medium level of seismic design (roughly corresponding to post-1980 codes in Southern Europe, such as the 1985 supplementary clauses of the Greek Seismic Code) and reasonable seismic detailing of R/C members, e.g. RC3.1HM (high rise, moderate code).
- High code: R/C buildings with an enhanced level of seismic design and ductile seismic detailing of R/C members according to the new generation of seismic codes (similar to Eurocode 8).

The available statistical data was not sufficient for distinguishing between all four subcategories of seismic design. Moreover, analysis of the damage statistics for Thessaloniki buildings after the 1978 Volvi earthquake (Penelis et al. 1989) has clearly shown that there was no reduction in the vulnerability of R/C buildings following the introduction of the first (rather primitive by today’s standards) seismic code in 1959. Even if this is not necessarily the case in all cities, differentiation between RC$\text{ixN}$ and RC$\text{ixL}$, as well as between
RCixM and RCixH is difficult, and judgement and/or code-type approaches are used to this effect. Three sets of analyses were finally carried out, for three distinct levels of design, ‘L’ (buildings up to 1985), ‘M’ (1986–1995) and ‘H’ (buildings designed to the 1995 and 2000 (EAK) Greek codes). The 1995 code (‘NEAK’) was the first truly modern seismic code (quite similar to Eurocode 8) introduced in Greece, and its differences from EAK2000 are minor and deemed not to affect the vulnerability of the buildings; hence buildings constructed from 1996 to date are classified as H. Differences (in terms of strength and available ductility) between N and L buildings, and M and H buildings are addressed in a semi-empirical way at the level of capacity curves (§5). All building classes studied herein are summarised in Table 1.

3. Inelastic analysis procedure

For all low-, moderate- and high-code R/C buildings, inelastic static and dynamic time-history analyses were carried out using the SAP2000N (Computers and Structures Inc. 2002) and the DRAIN-2000 (Kappos and Dymiotis 2000) codes, respectively. R/C members were modelled using lumped plasticity beam-column elements, while infill walls were modelled using the diagonal strut element for inelastic static analyses and the shear panel isoparametric element for inelastic dynamic analyses, as developed in previous studies (Kappos et al. 1998a). The R/C beam element of DRAIN-2000 follows the bilinear version of the Takeda model, which allows for stiffness degradation, but yield moment does not depend on axial load variation during the time-history analysis (hence it is taken to correspond to the gravity loading on columns). However, all ductility calculations are carried out by taking into account the axial load derived from dynamic analysis. Unlike column failures, beam failures are not considered to be critical when the aim is to prevent collapse. As also noted by Priestley and Calvi (1991), a beam that fails (i.e. the required rotational ductility $\mu_B$ exceeds the corresponding capacity $\mu_{0,u}$) loses its flexural capacity, but may continue to support the shear forces and span moments caused by the initial static loads. In DRAIN-2000, it is conservatively assumed that failure at one end of a beam implies failure of the entire member, and its stiffness and end moments are therefore reduced by 99% (a 100% reduction might lead to numerical problems) and correction is made for out-of-balance loads, as proposed by Dymiotis et al. (1999).

In total, 72 structures were addressed in the present study, but full analyses were carried out for 54 of them (N and L buildings were initially considered together, as discussed previously, but different pushover curves were finally drawn, see §5). To keep the cost of analysis within reasonable limits, all buildings were analysed as two-dimensional (2D) structures. Typical structures studied are shown in Figure 1. It is pointed out that, although the consideration of 2D models means that effects such as torsion due to irregularity in plan were ignored, previous studies (Kappos et al. 1998b) have shown that the entire analytical model (which also comprises the structural damage versus loss relationship) slightly underpredicts the actual losses of the 1978 Thessaloniki earthquake, from which the statistical damage data used in the hybrid procedure originate. Moreover, evaluation of that actual damage data has shown (Penelis et al. 1989) that plan irregularities due to asymmetric arrangement of masonry inwalls were far less influential than irregularities in elevation (soft storeys due to discontinuous arrangement of infills); the latter are directly taken into account in the adopted analytical models.

Using the DRAIN-2000 code, inelastic dynamic time-history analyses were carried out for each building type and for records scaled to several PGA values, until ‘failure’ was detected. 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 in several thousands of inelastic time-history analyses (the pseudoacceleration spectra of the 16 records are shown in Figure 2). The eight recorded motions are: four from

| Table 1. Specific building classes and design levels for R/C building analysis. |
|------------------|------------------|------------------|
| Type | Structural system | Height (number of storeys) | Seismic design level |
| RC1 | Concrete moment frames | (L)ow rise (1-3) | (N)/pre code |
| RC3 | Concrete moment frames with unreinforced masonry infill walls | (M)edium rise (4-7) | (L)ow code |
| 3.1 | Regularly infilled frames | (H)igh rise (8+) | (M)edium code |
| 3.2 | Irregularly infilled frames (pilotis) | (H)igh rise (8+) | (H)igh code |
| RC4 | RC dual systems (RC frames and walls) | (H)igh rise (8+) | (H)igh code |
| 4.1 | Bare frames (no infill walls) | (H)igh rise (8+) | (H)igh code |
| 4.2 | Regularly infilled dual systems | (H)igh rise (8+) | (H)igh code |
| 4.3 | Irregularly infilled dual systems (pilotis) | (H)igh rise (8+) | (H)igh code |
Figure 1. Configuration and dimensions (in m) of typical R/C building classes: (a) four-storey, irregularly infilled, R/C frame (RC3.2M type), (b) four-storey, regularly infilled, R/C dual system (RC4.2M) and (c) nine-storey, irregularly infilled, R/C dual system (RC4.3H).
the 1999 Athens earthquake (A299_T, A399_L, A399_T, A499_L), two from the 1995 Aegion earthquake (aigx, aigy) and two from the 2003 Lefkada earthquake (lefL, lefT). The eight synthetic motions were calculated for Volos (A4, B1, C1, D1) and Thessaloniki (I20_855, N31_855, I20_KOZ, N31_KOZ) sites (as part of microzonation studies). The use of PGA for the scaling of the 16 records is based on the need to use the same intensity measure for all systems considered; other quantities, such as the spectral acceleration for the fundamental period of a structure, could also be adopted within the framework of this procedure.

4. Estimation of economic loss using inelastic dynamic analysis

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. (1998b); for R/C members the cost is normalised to that of the most expensive technique (jacketing), and is calculated as a function of the largest rotational ductility ratio in the member, whereas, for brick masonry infills, the cost is normalised to that of replacing the infill and is calculated as a function of the interstorey drift at the storey where the infill is located. The total loss for the entire building is derived from empirical equations (calibrated against cost of damage data from Greece):

\[ L = 0.25D_c + 0.08D_p \quad (\leq 5 \text{ storeys}) \quad (1a) \]

and

\[ L = 0.30D_c + 0.08D_p \quad (6 - 10 \text{ storeys}) \quad (1b) \]

where \( D_c \) and \( D_p \) are the global damage indices (\( \leq 1 \)) 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% (architectural elements, all kinds of installations, etc. are not included) 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 (Dymiotis et al. 1999).
- 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 new set of failure criteria was recently proposed by Kappos et al. (2006), and resulted from the combination of ductility models previously developed by the senior author and his co-workers (e.g. Dymiotis et al. 1999), engineering judgement and evaluation of the results of a large number of inelastic time-history analyses for several building typologies, noting that, assuming first column failure means failure of the entire structure leads to unrealistic results, especially for buildings designed to modern seismic codes. Although they represent our 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 some limitations. For instance, although in most cases the value of earthquake intensity estimated to correspond to failure (damage state 5 in Table 3 later on) is reasonable, in some cases (in particular wall/dual structures, especially if designed to modern codes) PGAs associated with failure are unrealistically high and should be revised in future studies. Having said this, their influence in loss assessment is typically limited, since the scenario earthquakes do not lead to ground accelerations higher than about 1g.
The damage index for each building typology is estimated at each earthquake level (I or PGA) as the average $L$ value for the 16 input motions. The selection of the input motions is crucial for the results of the vulnerability analysis and it was found that the eight synthetic records used in most cases produced higher responses than the eight natural records. It is also noted that the use of the aforementioned failure criteria (yielding $L = 1$ for the corresponding analysis) can lead to average $L$ values (for a given PGA) that exceed 40%.

5. Damage assessment using pushover analysis

A pushover curve is a plot of a building’s lateral load resistance as a function of a characteristic lateral displacement (typically a base shear versus top displacement curve) derived from inelastic static (pushover) analysis. In order to facilitate direct comparison with spectral demand, base shear is converted to spectral acceleration and the roof displacement is converted to spectral displacement using modal properties and the equivalent Single Degree Of Freedom (SDOF) system approach, resulting in a ‘capacity curve’ in terms of spectral quantities (e.g. FEMA-NIBS 2003).

Pushover analyses were carried out for all low-code, moderate-code and high-code building models. No-code (or pre-code) buildings were assumed to have a strength that is 20% lower than those corresponding to low-code models, but the same displacement ductility factor ($S_{du}/S_{dy}$), reflecting the well-known fact that, in Greece, ductility was not an issue in seismic design prior to the 1985 revision of the seismic code.

Some typical pushover curves and their corresponding bilinear versions (derived on the basis of equal areas under the curves) are given in Figure 3; as shown in the figure, the equal areas are calculated up to the point where the first significant drop in strength (usually about 20%) occurs in the ‘complete’ pushover curve.

Bilinear pushover curves are constructed for each model building type and represent different levels of seismic design level and building performance. Each such curve is defined by two points: (1) the ‘yield’ capacity and (2) the ‘ultimate’ capacity. The yield capacity represents the strength level beyond which the response of the building is strongly nonlinear and is higher than the design strength, due to minimum code requirements, actual strength of materials are higher than the design value (mean values of concrete and steel strength were used in the nonlinear analyses) and, importantly, due to the presence of masonry infills (this influence is more pronounced in the case of frame systems), whenever such infills are present. The ultimate capacity is reached after the global structural system has developed a full mechanism and a 20% drop in strength has occurred, due to the fact that some members have failed, in the sense that they have exceeded their deformation capacity. It is emphasised that, due to the fact that the pushover curves used for the vulnerability assessment are bilinear versions of the actually calculated curves (Figure 3), a necessity arising from the fact that bilinear behaviour is considered in reducing the elastic spectrum to an inelastic one (or an equivalent elastic one for effective damping compatible with the energy dissipated by the inelastic system), the strength corresponding to the ultimate capacity generally does not coincide with the actual peak strength recorded during the analysis. Moreover, the yield capacity is not the strength of the building when first yielding of a member occurs. The proper way to ‘bilinearise’ a pushover curve is still a rather controversial issue, in the sense that different methods are more appropriate, depending on the objective of the specific analysis. It is worth recalling here that in the ATC-40 (1996) manual, where the capacity spectrum method is presented in detail, it is recommended to bilinearise the capacity curve with respect to the previously estimated target point, i.e. the bilinearised curve changes during each iteration, which is not a very convenient procedure.

Using standard conversion procedures (e.g. ATC 1996, FEMA-NIBS 2003), bilinear pushover curves ($V/W$ vs. $\Delta x/H_{tot}$, where $V$ and $\Delta x$ are the base shear and the roof displacement estimated from pushover analysis, $W$ the weight, and $H_{tot}$ the height, of the building) were transformed into capacity curves, i.e. plots of spectral pseudo-acceleration vs. spectral displacement ($S_a$ vs. $S_d$). The coordinates of the points describing the capacity curves are given for all R/C...
Table 2. Capacity curve parameters for frame buildings.

<table>
<thead>
<tr>
<th>Building type</th>
<th>Yield capacity point</th>
<th>Ultimate capacity point</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_{dy}$ (cm)</td>
<td>$S_{dy}$ (g)</td>
</tr>
<tr>
<td>RC1LL</td>
<td>1.15</td>
<td>0.187</td>
</tr>
<tr>
<td>RC1ML</td>
<td>3.28</td>
<td>0.17</td>
</tr>
<tr>
<td>RC1HH</td>
<td>4.31</td>
<td>0.125</td>
</tr>
<tr>
<td>RC1LM</td>
<td>1.14</td>
<td>0.398</td>
</tr>
<tr>
<td>RC2LM</td>
<td>2.72</td>
<td>0.213</td>
</tr>
<tr>
<td>RC1HM</td>
<td>6.83</td>
<td>0.238</td>
</tr>
<tr>
<td>RC1ML</td>
<td>4.45</td>
<td>0.746</td>
</tr>
<tr>
<td>RC1HH</td>
<td>4.9</td>
<td>0.427</td>
</tr>
<tr>
<td>RC1LL</td>
<td>13.34</td>
<td>0.245</td>
</tr>
<tr>
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<td>0.53</td>
<td>0.432</td>
</tr>
<tr>
<td>RC2ML</td>
<td>1.25</td>
<td>0.277</td>
</tr>
<tr>
<td>RC2HL</td>
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</tr>
<tr>
<td>RC2LM</td>
<td>0.59</td>
<td>0.49</td>
</tr>
<tr>
<td>RC2MM</td>
<td>1.39</td>
<td>0.274</td>
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<tr>
<td>RC2HM</td>
<td>2.26</td>
<td>0.266</td>
</tr>
<tr>
<td>RC2ML</td>
<td>0.973</td>
<td>0.975</td>
</tr>
<tr>
<td>RC2HH</td>
<td>1.64</td>
<td>0.538</td>
</tr>
<tr>
<td>RC2LL</td>
<td>4.26</td>
<td>0.34</td>
</tr>
<tr>
<td>RC2ML</td>
<td>0.88</td>
<td>0.201</td>
</tr>
<tr>
<td>RC3LL</td>
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<td>0.205</td>
</tr>
<tr>
<td>RC3HM</td>
<td>3.6</td>
<td>0.195</td>
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<tr>
<td>RC3MM</td>
<td>0.81</td>
<td>0.369</td>
</tr>
<tr>
<td>RC3HL</td>
<td>1.87</td>
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<tr>
<td>RC3MH</td>
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<td>RC3HH</td>
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</tr>
<tr>
<td>RC3LL</td>
<td>5.49</td>
<td>0.337</td>
</tr>
</tbody>
</table>

Table 3. Damage grading and loss indices (% of replacement cost).

<table>
<thead>
<tr>
<th>Damage state</th>
<th>Damage state label</th>
<th>Range of loss index</th>
<th>Central index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS0</td>
<td>None</td>
<td>0–0</td>
<td>0</td>
</tr>
<tr>
<td>DS1</td>
<td>Slight</td>
<td>&gt;0–1</td>
<td>0.5</td>
</tr>
<tr>
<td>DS2</td>
<td>Moderate</td>
<td>1–10</td>
<td>5.5</td>
</tr>
<tr>
<td>DS3</td>
<td>Substantial to heavy</td>
<td>10–30</td>
<td>20</td>
</tr>
<tr>
<td>DS4</td>
<td>Very heavy</td>
<td>30–60</td>
<td>45</td>
</tr>
<tr>
<td>DS5</td>
<td>Collapse</td>
<td>60–100</td>
<td>80</td>
</tr>
</tbody>
</table>

Since reduced spectra (inelastic, or elastic for effective damping ratios higher than 5%) are based on bilinear skeleton curves, it is not feasible (at least at this stage) to introduce multilinear pushover or capacity curves (i.e. including residual strength branches), hence it is suggested to tackle the problem as follows:

- Make use of the curves for which parameters are suggested for regularly infilled frames could be used.
- For higher $S_{du}$ values, analysis of the regularly infilled building should be repeated using the capacity curve for the corresponding bare one (RC1 or RC4.1); in some cases (particularly for pre-code or low-code buildings), it might be justified to use an $S_{du}$ value slightly reduced with respect to the bare frame, but this refinement is probably not warranted in the light of all the uncertainties involved.
- For pilotis buildings (RC3.2), it is conservatively suggested to assume that $S_{du}$ values, as reported in Table 2 are the actual ultimate values, except for the high-code case for which the procedure suggested for regularly infilled frames could be used.

Some example curves were shown in Figure 3 for R/C frame buildings designed to old codes (L); shown in the figure are (from top to bottom) the cases of infilled, pilotis and bare building, respectively. It is clear from these plots that, subsequent to failure of the ground storey infill walls, the strength of (fully) infilled frames becomes very close to that of the corresponding bare frame, while its ultimate deformation is somewhat lower. It is noted, though, that a ‘global type’ analysis that cannot fully capture local failure to R/C members due to interaction with infill walls, in principle cannot yield a reliable ultimate displacement for the structure; more work is clearly needed in this direction.

The pushover curves developed using the aforementioned procedure can be used for the derivation of fragility curves in terms of $S_{di}$, as presented in §6.3.
6. Derivation of fragility curves

One possibility for deriving vulnerability (fragility) curves is in terms of macroseismic intensity (I) or PGA; it is recalled herein that, as long as a certain empirical 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 (Penelis et al. 1989) used within the hybrid method was made using the relationship:

\[ \ln(\text{PGA}) = 0.74I + 0.03, \quad (2) \]

which is one of the most recent ones proposed for Greece (Koliopoulos et al. 1998) and is based on the statistical processing of a large number of Greek strong ground motion records. As is common with such relationships, it is associated with a significant scatter (the coefficient of determination \( R^2 = 0.57 \)); moreover, it is calibrated for intensities \( I \leq 8.5 \), hence it is not deemed appropriate for \( I \geq 9 \).

Assuming a lognormal distribution (a common assumption in seismic fragility studies), the conditional probability of being in, or exceeding, a particular damage state \( d_{si} \), given the PGA is defined by the relationship:

\[ P[d \geq d_{si}|\text{PGA}] = \Phi \left[ \frac{1}{\beta_{d_{si}}} \ln \left( \frac{\text{PGA}}{\overline{\text{PGA}}_{d_{si}} d_{si}} \right) \right], \quad (3) \]

where \( \overline{\text{PGA}}_{d_{si}} \) is the median value of PGA at which the building reaches the threshold of damage state \( d_{si} \) (see Table 3); \( \beta_{d_{si}} \) is the standard deviation of the natural logarithm of PGA for damage state \( d_{si} \) and \( \Phi \) is the standard normal cumulative distribution function.

Each fragility curve is defined by a median value of the PGA 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 below. Table 3 provides the best estimate values for the loss index ranges associated with each damage state, derived from previous experience with R/C structures (Kappos et al. 2006).

Median values for each damage state in the fragility curves were estimated for each of the 54 types of building systems analysed. The starting point for estimating these values is the plot of the damage index (calculated from inelastic time-history analysis, as described in §4) as a function of the earthquake intensity (PGA); some plots of this type are given in Figure 4 and they refer to buildings with frame systems designed to moderate codes (see §2). Several trends can be identified in the figure, for instance that the least vulnerable building is the fully infilled one, with the exception of very low PGA values, for which the loss is higher that in the pilotis (open ground storey) type; this is mostly due to damage in the masonry infills, which is accounted for in the loss model used (Kappos et al. 1998b) and in the case of the fully infilled frame is more widespread along the height of the building.

Lognormal standard deviation values (\( \beta \)) describe the total variability associated with each fragility curve. Three primary sources contribute to the total variability for any given damage state (FEMA-NIBS 2003), 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 Kappos (2001)), the variability associated with the capacity of each structural type and, finally, the variability of the demand imposed on the structure by the earthquake ground motion. The uncertainty in the definition of damage state, for all building types and all damage states, was assumed to be $\beta = 0.4$ (FEMA-NIBS 2003), the variability of the capacity for low-code buildings is assumed to be $\beta = 0.3$ and for high-code buildings, $\beta = 0.25$ (FEMA-NIBS), while the last (and most important) 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.

A first set of fragility curves was derived for the 54 R/C building classes, using the aforementioned procedure, based purely on the results of the incremental dynamic analyses. This last part of fragility analysis was carried out using in-house developed software (HyFragC), which permitted quick exploration of alternative approaches (sensitivity analysis). Examples of analytically derived fragility curves are given in Figure 5 for medium-rise, regularly infilled R/C frames designed to (a) low and (b) high codes. It is noted that the $\beta$ values are taken as constant for each building type; this constant value (estimated to be between about 0.6 and 0.7) is the average of the five values of $\beta$ corresponding to each of the five 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 where median values are closely spaced (e.g. see Figure 5a, DS3 and DS4).

6.1. Hybrid fragility curves based on statistical data for a single intensity

The hybrid approach is first applied using a combination of inelastic dynamic analysis and the database of the Thessaloniki earthquake of 1978 (Penelis et al. 1989), corresponding to an (average over the considered area) intensity $I = 6.5$, which is associated with a $\text{PGA} = 0.13g$, according to the adopted $I$–PGA relationship in Equation (2); 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 here as the ratio $L$ of repair cost to replacement cost (i.e. as a direct loss index), corresponding to this PGA is found for each building (a total of 5700 R/C buildings are included in the database). The Thessaloniki database is described in a number of previous publications (e.g. Penelis et al. 1989, Kappos et al. 1998b) and is still considered to be the most reliable seismic damage database in Greece for R/C buildings, since it was derived by in-situ work (carried out some years after the earthquake) on 50% of the building blocks in an area covering about half of the city (Penelis et al. 1989). It is worth pointing out here that other available databases typically suffer from inadequate knowledge of the total number of buildings, a sample of which is included in the database (Kappos 2001).

Median values of the PGA to be used in Equation (3) are then estimated from an updated (scaled) version of the PGA–damage index curves using a uniform (for all intensity/PGA points) ratio $\lambda = L_{\text{act}}/L_{\text{anl}}$

Figure 5. Purely analytical fragility curves (in terms of PGA) for medium-height R/C frames.
estimated at the $I = 6.5$ (PGA = 0.13) point, where $L_{\text{act}}$ is the actual (statistical) and $L_{\text{anl}}$ is the analytically calculated loss value (Figure 6). It is worth noting that the $\lambda$ ratios 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 (Kappos et al. 1998b). In this way, it is possible to establish a relationship between damage index and PGA for each building type (similar to the one shown in Figure 4, but now accounting for the empirical data as well, as described later), and consequently to assign a median value of the PGA to each damage state. Lognormal standard deviation values ($\beta$) are considered equal to the ones from the purely analytical approach.

The fact that the number of buildings for which loss data is available is insufficient for reliable statistical processing in the Thessaloniki 1978 database for several of the classes presented in Table 1, led to the need of using three different approaches (interpretations) for the combination of analytical results with statistical data (estimation of $\lambda$ ratios).

(1) For building classes with sufficient statistical data, the $\lambda$ ratio is estimated as above. If statistical data are limited, then the $\lambda$ ratio of the closer class with available data is used (e.g. RC3.2LL and RC3.1LL).

(2) A common $\lambda$ ratio for all building classes of the same height is used, defined as:

$$\lambda = \frac{\sum_{i=1}^{n} L_{\text{anl},I=6.5} N_i}{\sum_{i=1}^{n} N_i}.$$  

(4) The $\lambda$ ratio is defined as the first approach, but a common loss index $L_{\text{anl},I=6.5}$ (at point $I = 6.5$) is assumed for all building classes of the same height, defined as:

$$L_{L_{\text{aver}}} = \frac{\sum_{i=1}^{n} L_{\text{act},I=6.5} N_i}{\sum_{i=1}^{n} N_i},$$  

5

where $L_{\text{act},I=6.5}$ is the actual (statistical) loss value at a point $I = 6.5$ for building class $i$; $L_{\text{anl},I=6.5}$ is the analytically calculated loss value at a point $I = 6.5$ for building class $i$; $i = 1, 2, \ldots, n$ is the building class with sufficient available statistical data; and $N_i$ is the number of buildings of class $i$ in the database.

It is noted that the three approaches are only applied to the low-code building classes, since no moderate- or high-code buildings were present when the Thessaloniki 1978 earthquake occurred. The same $\lambda$ ratios estimated for the low-code building classes were used for the corresponding (i.e., having the same structural system, height, infills arrangement) moderate- and high-code classes (e.g. $\lambda_{\text{RC3.1ML}} = \lambda_{\text{RC3.1MM}} = \lambda_{\text{RC3.1MH}}$). Some examples of hybrid fragility curves are presented in Figure 7. The effect on the resulting fragility curves of the way statistical data are interpreted in the hybrid approach appears to be rather significant, particularly for the higher damage states. Also, as anticipated, the effect of seismic design is significant; buildings designed to only a moderate seismic code are seen to be substantially less vulnerable than buildings designed to low code, pointing to the importance of using some basic seismic design rules (such as some basic form of capacity design and ductility), even if these rules are not strictly in compliance with modern code provisions.

6.2. Hybrid fragility curves based on statistical data for multiple intensities

The use of uniform values for the $\lambda$ ratio (for all intensity/PGA levels) to update the PGA–damage index curves may be problematic and lead to unrealistic results, especially if its values are significantly different from 1.0 (e.g., $\lambda < 0.5$ or $\lambda > 2.0$) and/or the actual data correspond to a very low (or, less often, to a very high) intensity. This is due to the strongly nonlinear nature of the relationship between intensity and structural damage (Kappos 1997) and is one of the key reasons why three different interpretations were used for the combination of the Thessaloniki damage data with the analytical results.

Provided that actual (empirical) results are available for more than one intensity/PGA levels (points in the damage index versus PGA relationship of Figures 4 and 6), different $\lambda$ values can be used at each point; nevertheless, interpolation and extrapolation are still
not straightforward. Together with the extension of the hybrid approach to the case of statistical data for multiple intensities, a new concept is introduced in the procedure, that of weighting factors to account for the reliability of the statistical data. The weighting concept can also be used in the case of data for a single intensity, in lieu of the somewhat arbitrary interpretations presented in §6.1.

Having established analytically the loss index $L_i$, the final value to be used for each PGA in the fragility analysis depends on whether an empirical value is available for that PGA or not, i.e. (i) if the actual (statistical–empirical) loss value at a point $i$ ($PGA_i = PGA_a$), $L_{act,i}$ is available in the database, the final value to be used is:

$$L_{fin,i} = w_{1,i}L_{act,i} + w_{2,i}L_{anl,i}(w_{1,i} + w_{2,i} = 1), \quad (6a)$$

where $L_{anl,i}$ is the analytically calculated loss value (Figure 4) for that PGA; and $w_{1,i}$ and $w_{2,i}$ are weighting factors that depend on the sample size and the reliability of the empirical data available at that intensity. If $L_{act,i}$ is based on more than about 60 buildings with reliable data, $w_{1,i}$ equal to about 1 is recommended, if it is based on six buildings or less, $w_{1,i}$ should be taken as zero (or nearly so). The ratio $l_i = L_{fin,i}/L_{anl,i}$ at point $i$ is:

$$l_i = w_{1,i}(L_{act,i}/L_{anl,i}) + w_{2,i}. \quad (6b)$$

(ii) if the actual loss value at a point $j$ ($PGA_j$), $L_{act,j}$ is not available in the database, new actual loss values, as well as new weighting factors, are estimated using linear interpolation between points $i$ and $k$ with available data ($PGA_i < PGA_j < PGA_k$).

Clearly, this is an interpolation scheme that aims to account (in a feasible way) for the strongly nonlinear relationship between intensity and damage. In the common case, that $L_{act}$ is available at one or very few points, the scheme should be properly adapted, as discussed subsequently.

A first application of this procedure is shown here for one of the building classes studied, RC3.1LL (regularly infilled, low-rise, R/C frames designed to low codes). Along with the Thessaloniki 1978 earthquake database (that includes 390 RC3.1LL

Figure 7. Hybrid fragility curves (in terms of PGA) for several R/C building classes, based on the Thessaloniki 1978 database.
buildings), the database for the Municipality of Ano Liosia from the 1999 Athens earthquake (Kappos et al. 2007) was also used (it includes 434 RC3.1LL buildings). The macroseismic intensity value for the Municipality of Ano Liosia was I = 9.0, according to the National Observatory of Athens; for this I, Equation (2) (calibrated for I ≤ 8.5) yields a PGA of 0.82g, and this value is used here (mainly because, in this way, the case studied provides an opportunity for testing the new procedure in a difficult case, whereby actual damage data points lie both above and below the analytically derived curve), although the authors believe that a lower value would be appropriate. It is worth noting, in this respect, that the actual (empirical) loss index value estimated from the Ano Liosia database (L_{act} = 34.5%) is very close to the analytically calculated value (L_{anl,I = 8.0} = 35.4%) for I = 8.0 (PGA = 0.35g).

The weighting factor pairs are taken as w_{1,I = 6.5} = 0.90, w_{2,I = 6.5} = 0.10 for the Thessaloniki database, since the sample is considered to be sufficient and reliable (direct loss index values were available in the database) and w_{1,I = 9.0} = 0.60, w_{2,I = 9.0} = 0.40 for the Ano Liosia database, since although the sample size is considered to be sufficient, loss values were implicitly estimated (Kappos et al. 2007) from average repair costs for buildings with green, yellow and red tags assigned during the post-earthquake assessment. The analytical PGA versus damage (loss) index curve is updated, as shown in Figure 8, which also shows the median values of the PGA assigned to each damage state (using the definitions of Table 3).

All empirical relationships between PGA and I are clearly characterised by large scatter. A first attempt to quantify the effect of this scatter on the results of the hybrid approach was carried out using an alternative relationship (Theodulidis and Papazachos 1992), which was derived using a database that consists of 105 horizontal components from 36 shallow earthquakes in Greece of magnitude 4.5 to 7.0:

\[
\ln(\text{PGA}) = 0.28 + 0.67I + 0.42S + 0.59P,
\]

where \(S = 0\) at alluvium sites and \(S = 1\) at rock sites; and \(P = 0\) for 50 percentile values and \(P = 1\) for 84 percentile values. As can be seen in Figure 8, differences of about 20% were found in the damage evolution curve, the 1992 relationship (Equation (7)) resulting in higher predicted damage than the 1998 one (Equation (2)); this is deemed commensurate with the remaining uncertainties involved in the fragility assessment procedure.

The fragility curves derived from the weighted hybrid approach (using Equation (2)) are shown in Figure 9, together with the corresponding ones derived from the procedure of §6.1 (first interpretation), and it is clear that the effect of different procedures is quite significant. The weighted approach seems to lead to more realistic results for the RC3.1LL class, as the single intensity procedure looks rather conservative (\(P[ds > ds_5| \text{PGA} = 0.50] = 72.6\%\)). This is usually the case when the \(\lambda\) ratio is significantly greater than 1.0 at the Thessaloniki 1978 intensity point (I = 6.5). The exact opposite behaviour (non-conservative fragility curves according to the §6.1 procedure) is observed for building classes with \(\lambda\) ratios significantly smaller than 1.0. The weighted hybrid approach manages to overcome these problems, provided that sufficient statistical data are available for high (I ≥ 8.0) intensity values.

![Figure 8. PGA-damage index relationship for the RC3.1LL building class. Calculation of hybrid median PGA values using empirical weighting factors, and two different empirical equations for the PGA–I relationship.](image1)

![Figure 9. Hybrid fragility curves for the RC3.1LL building class using statistical data for multiple intensities and empirical weighting factors (full lines) versus the corresponding ones using the Thessaloniki 1978 database (dashed lines).](image2)
6.3. Fragility curves in terms of $S_d$

The aforementioned fragility curves in terms of the PGA were also used to derive additional curves, this time in terms of $S_d$, which are necessary for fragility assessment using approaches such as HAZUS (FEMA-NIBS 2003). The procedure adopted was to transform the median PGA values to corresponding median $S_d$ values, using an appropriate spectrum, the bilinear capacity curves derived as discussed in §5, and the, now well-known, capacity spectrum approach, as illustrated in Figure 10 (where $T_{el}$ is the ‘elastic’ period of the structure calculated from the bilinear capacity curve, $\mu$ is the ductility for which the inelastic spectrum is drawn, and $\mu_{TR}$ the ductility at the performance point).

For the present application of the methodology, it was decided to use the mean spectrum of the microzonation study of Thessaloniki (Anastasiadis et al. 2001), shown in Figure 11, together with the Greek Code (EAK) spectrum. Clearly other options are also available, the most conservative one being to use the seismic code design spectrum, which has been found to overestimate seismic actions (particularly displacements) for medium and long period structures (Athanassiadou et al. 2007).

Two examples of $S_d$-based fragility curves are given in Figure 12 (nine-storey infilled frames, designed to low codes) using the microzonation study of (a) the Thessaloniki (a) and (b) the EAK2000 spectra. It should be noted that the median $S_d$ values (derived from the corresponding PGA ones) are based on the damage state definitions presented in Table 3, used for the PGA approach, i.e. damage states were not defined

**Figure 10.** Example of application of the capacity spectrum method (RC4.1LL class).

**Figure 11.** Mean spectrum of the microzonation study of Thessaloniki.

**Figure 12.** $S_d$-based fragility curves for the RC3.1LL building class using: (a) the Thessaloniki microzonation and (b) the Greek code (EAK2000) spectra.
directly in terms of displacements. In Figure 13, the location of the displacements defining the damage states of building type RC3.1ML are shown drawn on the capacity curves of the corresponding building classes (see also §5). It is clear that, by applying this indirect procedure, damage thresholds based on the economic loss indices presented in Table 3 are quite different (generally, more conservative) than those of HAZUS that uses an alternative approach for the definition of damage states based on the capacity curves of the structures.

A more detailed discussion of the impact that the type of fragility curve used for a vulnerability assessment study has on the end results (loss scenario) is given by Pitilakis et al. (2004), wherein the damage and loss scenario for Thessaloniki, developed using both approaches, is presented.

7. Conclusions

This paper has tackled a number of issues relating to vulnerability and loss assessment, with particular emphasis on the situation in Southern Europe. A classification scheme that is deemed appropriate for the building stock in this area has been presented, aiming at an adequate description of the R/C buildings that currently dominate the built volume in this area.

The key idea of the hybrid approach to seismic vulnerability assessment is the combination of damage statistics (empirical data) with results from inelastic analysis; this is an approach that clearly differs from most other procedures, among which the well-known procedure adopted by HAZUS, wherein fragility curves are based directly on inelastic (static) analysis, and the only empirical component in their derivation is the definition (by judgement) of the damage state thresholds. The key empirical parameter was the cost of repair of a damaged building; this is a particularly useful parameter, but reliable data are not always available on it, which means that other parameters (structural damage indices) could certainly be explored within the broader frame of the hybrid approach.

The type of assumption made for the functional form of the fragility curve is also a key one, but the current trend worldwide seems to be towards adopting the lognormal cumulative distribution function; the determination of damage medians and the variabilities associated with each damage state can be carried out using the procedures described in HAZUS, or the alternative ones suggested herein. It is noted, however, that values of the variabilities proposed in HAZUS should not be adopted blindly if the analytical procedure used is not the one based on the capacity spectrum.

Regarding the two different types of fragility curves that can be used, PGA-based curves offer a number of advantages, but also ignore, to an extent that depends on the spectral characteristics of the motions considered for deriving the fragility curves and their relationship to the characteristics of the scenario motions, the possibly lower damageability of motions with high PGA and spectra peaking over a very narrow band and/or with very short duration (both these characteristics are more or less typical in strong motions recorded in Greece). The $S_d$-based curves take into account the spectral characteristics of the motion, but further research is needed as to what type of spectra should be used in this respect.

Finally, the new, extended version of the hybrid approach presented here, that allows both for incorporating damage data for multiple intensities and for weighting analytical and statistical–empirical data points, seems to be promising. It is clear, though, that further research is needed in this direction, notably for the calibration of the weighting factors used, which, for the time being, are purely empirical (based on our best judgement).

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