Assessment of the seismic vulnerability of unreinforced masonry buildings

Gr. G. Penelis, A. J. Kappos & K. C. Stylianidis

Aristotle University of Thessaloniki Dept. of Civil Engineering, Greece

Abstract

A methodology for seismic vulnerability analysis of URM buildings is presented. It includes the estimation of capacity curves for typical building classes, as well as vulnerability (fragility) curves in terms of peak ground acceleration (PGA). The methodology for developing vulnerability curves is based on a hybrid approach, combining statistical data with appropriately processed results from nonlinear static analyses that permit extrapolation of statistical data to intensities for which no data are available. The data used are statistical, derived from Greek earthquakes, the Thessaloniki 1978 and the Aegion 1995 events, with some additional data from the Pyrgos 1993 earthquake used for comparison purposes. The databases of the first two earthquakes are briefly presented and processed using a filtering technique. The resulting hybrid vulnerability curves correlate PGA to the probability that a building type exceeds a particular damage state.

1. Introduction

The so-called 2nd level approach (Penelis et al. 2002) for seismic vulnerability analysis involves fragility curves in terms of spectral quantities (displacements or accelerations); these quantities are determined from a convolution of the demand spectrum and the resistance curve of a building. The work on unreinforced masonry (URM) buildings presented herein focuses on two distinct objectives: the development of characteristic ('prototype') pushover and capacity curves for typical URM buildings, and the development of vulnerability (fragility) curves using PGA to describe the earthquake input.

Both objectives require a series of nonlinear static analyses of appropriately selected URM building types in order to produce reliable capacity curves as well as to utilise the hybrid vulnerability approach developed by the Aristotle
University (AUTH) team to compensate for the lack of actual (statistical) damage data for a number of earthquake intensities.

2. Building typologies considered

The work presented herein refers mainly to simple stone masonry buildings and brick masonry buildings with reinforced concrete floor-slabs, which are by far the most common types in Thessaloniki, as well as in the rest of Greek cities (see also Penelis et al. 2002). These two main categories are further subdivided into single-storey, two-storey and three-storey buildings; the specific buildings analysed within the present study are described in section 3.2.1.

3. Construction of capacity curves

3.1 Basic concepts

A pushover curve is a plot of a building’s lateral load resistance as a function of the lateral displacement. It is commonly presented as a plot of base shear (preferably normalised with respect to building weight) versus building displacement at roof level (preferably normalised with respect to total building height). Two control points that define a bilinearised pushover curve correspond to the “Yield Capacity” and the “Ultimate Capacity”.

Pushover curves can be converted to so-called capacity curves, based on the equivalent SDOF system approach (see e.g. FEMA 1997); capacity curves are constructed based on estimates of engineering properties that affect the design, yield and ultimate capacities of each model building type.

The calculation of prototype pushover and capacity curves is achieved through a series of nonlinear static (pushover) analyses with several variations in the material properties and the building geometry in order to establish an adequate level of confidence in the results.

The method adopted herein for the pushover analysis of URM buildings uses equivalent frame models and concentrated non-linearity at the ends of the structural elements, with a view to simplifying this otherwise cumbersome (for URM buildings) procedure. The non-linearity is simulated with nonlinear rotational springs, whose constitutive law is defined by the moment – rotation curve of each element accounting for both flexure and shear (Kappos, Penelis and Drakopoulos 2002).

For the development of the inelastic M-θ curves due to flexure, the following assumptions have been made:

- Zero masonry tensile strength
- Parabolic distribution of compression stresses
- Bernoulli compatibility (plane sections remain plane) up to failure
- Compressive deformation at failure $\varepsilon_0 = -2\%$
- Compressive strength ($f_m$) of masonry uniform, defined by the strength of bricks and mortar
- Modulus of elasticity $E_m=550f_m$ (FEMA 1997)
The nonlinear shear behaviour has been modelled using a Mohr–Coulomb failure criterion for the definition of shear strength and a statistical analysis of experimental results for defining shear deformations (Kappos et al. 2002).

As validation of the results, the analytical simulation of two test cases, a URM wall tested in Pavia (Calvi et al. 1996) and a URM building tested at ISMES, Bergamo, have been presented in other publications (Kappos A.J., Penelis, Gr.G., and Drakopoulos, C., 2002).

3.1.1 Building types analysed

As mentioned previously, analysis of several different URM buildings has been performed. More specifically the generic structure considered followed the layout shown in figure (1) and was used for one to three storey URM buildings. This layout corresponds to a typical residential building roughly at the threshold of EC8 category ‘simple buildings’.

Two different material properties were used for all the above buildings types: Material A with $f_{wm}=1.5$ MPa and Material B with $f_{wm}=3.0$ MPa; for the Young’s modulus $E=550f_{wm}$ was assumed, as mentioned previously.

All the aforementioned combinations of parameters result in a total of thirty-six different building types, which were analysed using static nonlinear analysis in order to derive the resistance (pushover) and capacity curves.

3.1.2 Results

The thirty-six different pushover curves resulted from the analysis of the alternative models considered, averaged per number of stories, are shown in figure 2. In figures 2(a) and (b) these curves are compared with experimental data from Pavia and ISMES and with the proposed curves in HAZUS (FEMA-NIBS 1999). It is clear that the capacity curves adopted by HAZUS are out of scale in comparison with the curves developed herein, especially those for the low-rise building type. The large discrepancy between HAZUS and the proposed curves lies not in strengths (which are similar if ultimate strength in HAZUS is considered) but in displacements, which are substantially lower in the present
analysis, as well as in the tests used for comparison purposes. In table 1 all the analytical features presented graphically in figures 2(a) and 2(b) are presented and compared in a tabular format together with available Italian test data (Calvi et al. 1996, Benedetti et al. 1998) and other proposals (Calvi 1999).

![Fig. 2: Pushover curves and comparison with experimental results and HAZUS curves](image)

On the basis of table 1, the comment made with respect to figure 4 is further emphasized, since all suggested values are consistent, except maximum displacements (drifts) suggested by HAZUS which are outliers with regard to both the present analysis and the test data. It is noted that, to the authors’ best judgement, the only factor that was not considered in the analyses (and also in the tests) and could contribute to increased displacements is foundation compliance; this does not explain, though, the aforementioned large discrepancies.

In view of the above discrepancies, it is essential to discuss the issue of capacity curves for URM buildings and decide to adopt the values that are more consistent with European data.

Table 1: Comparison of analytical results with HAZUS, tests, and Italian data

<table>
<thead>
<tr>
<th>Storeys</th>
<th>Period [sec]</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HAZUS99</td>
<td>Calvi et al.</td>
</tr>
<tr>
<td>1</td>
<td>0.35</td>
<td>0.123</td>
</tr>
<tr>
<td>2</td>
<td>N/A</td>
<td>0.153</td>
</tr>
<tr>
<td>3</td>
<td>0.5</td>
<td>0.233</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storeys</th>
<th>Δu</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HAZUS99</td>
<td>Calvi et al.</td>
</tr>
<tr>
<td>1</td>
<td>1.800%</td>
<td>0.300%</td>
</tr>
<tr>
<td>2</td>
<td>N/A</td>
<td>0.300%</td>
</tr>
<tr>
<td>3</td>
<td>0.613%</td>
<td>0.300%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storeys</th>
<th>Δu</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HAZUS99</td>
<td>Calvi et al.</td>
</tr>
<tr>
<td>1</td>
<td>0.20</td>
<td>x</td>
</tr>
</tbody>
</table>
3.1.3 Proposed capacity curves
From the previously presented analyses, capacity curves (in HAZUS format) have been derived for one, two and three storey URM buildings. The corresponding parameters for these curves are given in table 2.

Table 2: Capacity curve parameters

<table>
<thead>
<tr>
<th>BUILDING</th>
<th>Height (m)</th>
<th>$\delta_0$ (C/(\xi))</th>
<th>$\delta_0$ (C/(\xi))</th>
<th>$\delta_0$ (C/(\xi))</th>
<th>$\delta_0$ (C/(\xi))</th>
<th>m [g]</th>
<th>T (sec)</th>
<th>$D_r$ (mm)</th>
<th>$D_s$ (mm)</th>
<th>(\mu)</th>
<th>(\gamma_r) (%)</th>
<th>(\gamma_s) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-storey</td>
<td>3.06</td>
<td>0.28</td>
<td>0.29</td>
<td>810907</td>
<td>262.4</td>
<td>0.12</td>
<td>1.05</td>
<td>5.76</td>
<td>5.46</td>
<td>0.034</td>
<td>0.188</td>
<td></td>
</tr>
<tr>
<td>2-storey</td>
<td>6.19</td>
<td>0.16</td>
<td>0.19</td>
<td>296295</td>
<td>524.9</td>
<td>0.29</td>
<td>3.12</td>
<td>14.90</td>
<td>4.78</td>
<td>0.050</td>
<td>0.241</td>
<td></td>
</tr>
<tr>
<td>3-storey</td>
<td>9.32</td>
<td>0.11</td>
<td>0.13</td>
<td>155501</td>
<td>787.3</td>
<td>0.49</td>
<td>6.40</td>
<td>22.33</td>
<td>3.49</td>
<td>0.069</td>
<td>0.239</td>
<td></td>
</tr>
<tr>
<td>MEAN</td>
<td>6.19</td>
<td>0.18</td>
<td>0.20</td>
<td>420901</td>
<td>524.9</td>
<td>0.30</td>
<td>3.52</td>
<td>14.33</td>
<td>4.58</td>
<td>0.051</td>
<td>0.223</td>
<td></td>
</tr>
</tbody>
</table>

4. Construction of vulnerability curves

4.1 General
The 2\textsuperscript{nd} level methodology aims at the calculation of practical expressions, which correlate the earthquake intensity (expressed here in terms of PGA) to a descriptor of the building’s damage state (damage level, repair cost, etc). In order to produce vulnerability (fragility) curves that describe more accurately the expected damage for several excitation intensities, a ‘hybrid’ approach has been used herein. More specifically, the nonlinear analysis presented in section 3 has been combined with the available damage statistics from actual events (Thessaloniki, 1978 and Aegion, 1995).

4.2 Hybrid methodology
The so-called “hybrid” approach to seismic vulnerability (Kappos et al. 1998, Kappos 2001) has been developed in recognition of the fact that reliable statistical data for seismic damage are quite limited and typically correspond to a limited number of intensities.

In Greece, the only reliable damage data available are those from the 1978 earthquake, derived during a previous project (Penelis et al. 1989). Damage data collected in Greece after more recent earthquakes (Kalamata 1986, Pyrgos 1993, Patras 1993, Aegion 1995), albeit valuable, are generally not in a form that economic damage statistics can be assessed for a representative set of buildings. What usually happens is that the collected data concerns only buildings that have been inspected for a second time (i.e. after the initial post-earthquake rapid
screening) and/or wherein some post-earthquake intervention (repair, strengthening, etc.) has taken place; furthermore, the extent of the geographical area, hence the total building stock to which the data refers, is often unclear.

The basics of the mechanical approach to vulnerability assessment are described in Kappos’ contribution to the EAEE WG3 report (Dolce et al. 1995), while the procedure to apply it for deriving damage probability matrices is described in Kappos et al. (1998).

4.3 Damage states

Five different damage levels (states) are proposed herein, defined according to the damage index shown in table 3. The (economic) damage index is the ratio of cost of repair to cost of replacement of a building and it is deemed suitable for vulnerability and loss assessment purposes, since it is a “monetary” index.

Table 3: Definition of damage states (levels)

<table>
<thead>
<tr>
<th>Damage level</th>
<th>Definition</th>
<th>Range of damage index</th>
</tr>
</thead>
<tbody>
<tr>
<td>No damage</td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Slight damage</td>
<td></td>
<td>0-5</td>
</tr>
<tr>
<td>Moderate damage</td>
<td></td>
<td>5-20</td>
</tr>
<tr>
<td>Extensive damage</td>
<td></td>
<td>20-50</td>
</tr>
<tr>
<td>Very heavy damage &amp; collapse</td>
<td></td>
<td>50-100</td>
</tr>
</tbody>
</table>

To correlate these damage states to an analytical expression of damage, the building damage index was expressed as a function of yield and ultimate displacement of each building as shown in figure 3; this approach is deemed more versatile than previous ones (e.g. Calvi 1999) based on fixed values of drift ratio for all building types. Using these definitions, the expected damage state of a particular building class can be assessed by combining the relevant capacity curve with the displacement corresponding to a given earthquake intensity.

Fig. 3: Damage in URM buildings, as a function of roof displacement
4.4 Vulnerability curves

It is well documented in the current literature that such curves can be described by (cumulative) normal, lognormal, beta or other distribution, provided that sufficient data is available. The most common problem when applying a purely empirical approach is the unavailability of (reliable) data for a sufficient number of 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 9 or greater are rare, at least for S. Europe, so there are hardly any 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. This makes the selection of an appropriate cumulative distribution very unreliable since the curve fit error is significant and the curve shape not as it would be expected.

In order to overcome these problems the Auth team is following the hybrid procedure since all the required analytical simulations for the hybrid methods have already been carried, while the very nature of the 2nd level approach involves nonlinear analysis.

4.5 Utilisation of available damage databases

Using the information from the Thessaloniki and Aegion databases for deriving damage probability matrices (DPMs) and vulnerability curves, it is essential to firstly define the PGA assigned to each earthquake.

Based on information reported in the literature (Kappos et al. 1998, Lekidis et al. 1999), the Thessaloniki earthquake gave an intensity 7 in the studied area, while the Aegion earthquake resulted in an intensity of 8 (in the town of Aegion). The spectral quantities required for 2nd level fragility curves were derived assuming that the spectra of the available records are representative of the ground motion in the corresponding areas.

To establish a link between PGA and intensity (in terms of which empirical data are available) the statistical relationship defined for Greek earthquakes by Theodulidis and Papazachos (1992) was used, which resulted in the PGA values of 0.143g for the Thessaloniki event and 0.267g for the Aegion one.

4.6 2nd level vulnerability curves in terms of PGA

The DPMs corresponding to PGA values lower than those from the Thessaloniki event were calculated by scaling down the Thessaloniki database, while the ones that correspond to higher than the Aegion event are calculated by scaling up the Aegion database. The scale factor was calculated by using the purely analytical DPMs for all PGA's.

Using the aforementioned hybrid procedure, cumulative DPMs such as those shown in table 4 have been calculated for the URM building classes of interest.
Table 4: Cumulative hybrid DPM for two-storey stone URM

2-storey stone URM | 2-storey brick URM
---|---
GA | 0.04 | 0.06 | 0.11 | 0.2 | 0.39 | 0.72 | 0.04 | 0.06 | 0.11 | 0.2 | 0.39 | 0.72

| | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% |
| | 0% | 19% | 36% | 36% | 60% | 60% | 0% | 1% | 12% | 12% | 59% | 59% |
| | 0% | 0% | 19% | 34% | 58% | 58% | 0% | 0% | 1% | 11% | 56% | 56% |
| | 0% | 0% | 0% | 19% | 33% | 33% | 0% | 0% | 0% | 1% | 35% | 35% |
| | 0% | 0% | 0% | 11% | 15% | 15% | 0% | 0% | 0% | 0% | 9% | 9% |

Vulnerability (fragility) curves were then derived by fitting the lognormal cumulative distribution function to the aforementioned cumulative DPMs. The parameters of the lognormal distribution functions were calculated for each building type and damage state.

Using these parameters, the fragility curves shown in figures 4 and 5 are plotted. Alternative curves in terms of spectral displacements have also been derived using again the hybrid approach and are presented elsewhere (Penelis et al. 2002).

Fig. 4: Lognormal cumulative distribution curves for single-storey and two-storey stone masonry buildings

Fig. 5: Lognormal cumulative distribution curves for single-storey and two-storey brick masonry buildings
5. Conclusions

The methodology adopted for the development of the 2nd level analysis of URM buildings has been presented herein. The work included two independent phases: the definition of capacity curves for URM building typologies common in Greece, and the definition of 2nd level vulnerability (fragility) curves correlating PGA to the probability of a building type to exceed a particular damage state, using a hybrid methodology which combines actual statistical data collected from past earthquakes with inelastic analysis for the intensities for which there are no data available. The statistical data used are from Greek earthquakes, the Thessaloniki 1978 and the Aegion 1995 events, with some additional data from the Pyrgos 1993 earthquake.

The results presented herein include capacity curves (in HAZUS format) for single-storey and two-storey URM buildings of either stone or brick masonry. Moreover, using the aforementioned hybrid approach, new fragility curves for standard URM building classes have been constructed, in terms of PGA. These are based both on a limited number of analyses (12 for each class), and on a limited number of ground motions (Thessaloniki 1978 and Aegion 1995 records), hence they should be further checked and verified.

The use of PGA rather than spectral displacement in 2nd level vulnerability curves, although conceptually less attractive, offers the advantage that a more direct correlation with statistical data is possible and there is no need to introduce additional assumptions (and uncertainty) regarding the spectral shape. Clearly, reliable relationships (based on local data) correlating intensity and PGA are necessary in the proposed hybrid methodology.

References


Karantoni, F. V., Bouckovalas, G., 1997. Description and analysis of building damage due to Pyrgos, Greece earthquake, Soil Dynamics & Earthquake Engineering, 16(2), 141-150.


