Engineering Structures 56 (2013) 343–356

Contents lists available at SciVerse ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Extending displacement-based earthquake loss assessment (DBELA) for the computation of fragility curves

V. Silva [a,b,*], H. Crowley [b], R. Pinho [c], H. Varum [a]

[a] University of Aveiro, Campus Universitário de Santiago, Aveiro, 3810, Portugal
[b] EUCENTRE, via Ferrata 1, Pavia, 27100, Italy
[c] University of Pavia, via Ferrata 1, Pavia, 27100, Italy

ARTICLE INFO

Article history:
Received 12 October 2012
Revised 25 April 2013
Accepted 29 April 2013

Keywords:
Seismic risk
Loss assessment
Fragility functions
DBELA
RC Turkish buildings

ABSTRACT

This paper presents a new procedure to derive fragility functions for populations of buildings that relies on the displacement-based earthquake loss assessment (DBELA) methodology. The recent developments in this methodology are also presented herein, such as the development of new formulae for the calculation of the yield period or the consideration of infilled frame structures. In the fragility method proposed herein, thousands of synthetic buildings have been produced considering probabilistic distributions describing the variability in their geometrical and material properties. Then, their nonlinear capacity has been estimated using the DBELA method and their performance against a large set of ground motion records has been calculated. Global limit states are used to estimate the distribution of buildings in each damage state for different levels of ground motion, and a regression algorithm is applied to derive fragility functions for each limit state. The proposed methodology is demonstrated for the case of ductile and non-ductile Turkish reinforced concrete buildings with and without masonry infill walls, and compared with results obtained using nonlinear dynamic procedures and with the results from previous studies.

1. Introduction

Fragility functions representing the probability of exceeding a set of damage states conditional on a level of ground motion are a fundamental component to describe the physical vulnerability of a population of buildings. The increase in the demand for reliable and more accurate loss estimations has triggered the development of fragility functions based on analytical/mechanical approaches which tend to provide a better representation of the structural behaviour of the building typologies. As discussed by Rossetto and Elnashai [41], there is no unique methodology for the development of fragility functions and therefore, each approach will have its limitations and advantages. Several methodologies ([45,18,3,19]; amongst others) have been proposed with different levels of simplification and efficiency in the past years. However, it is well established that one of the main drawbacks of any analytical methodology is the required computational and modelling effort. For this reason, a simplified methodology is proposed in this study.

The so-called DBELA methodology (e.g. [13,7]) is employed to estimate the nonlinear capacity of thousands of reinforced concrete (RC) frames randomly generated and the associated demand from a large set of ground motion records. The fact that several synthetic buildings and ground motion records are used in the calculations allows the consideration of the material and geometrical uncertainties, as well as (to some extent) the record-to-record variability. These calculations are performed within a probabilistic framework and therefore, the parameters that define the fragility functions (i.e. logarithmic mean and logarithmic standard deviation) are also described by a probabilistic distribution, which permits the propagation of the uncertainty in the vulnerability to the risk analysis. This procedure proved to provide a good balance between computational efficiency and reliability, allowing a quick and simple assessment of the physical vulnerability of many different building typologies (e.g. reinforced concrete frames or shear walls, masonry buildings with concrete or timber slabs).

This methodology is applied herein to estimate the statistics of fragility functions for real Turkish reinforced concrete frames with and without masonry infills walls. Then, these results are compared with previous studies, as well as with results obtained using complex nonlinear dynamic analysis, showing that despite the simplicity of the proposed methodology, satisfactory results are still attained.

* Corresponding author at: University of Aveiro, Campus Universitário de Santiago, Aveiro, 3810, Portugal. Tel.: +351 234370938.
E-mail address: vitor.s@ua.pt (V. Silva).

0141-0296/$ - see front matter © 2013 Elsevier Ltd. All rights reserved.
http://dx.doi.org/10.1016/j.engstruct.2013.04.023

2. DBELA fragility function calculator

Since the initial publications of the DBELA methodology [24,13], several improvements have been suggested, such as the development of new period/height relationships or the consideration of other building typologies. The new developments that concern the assessment of reinforced concrete frames have been compiled and are described in this section. Then, the proposed methodology to derive fragility functions is comprehensively described. A procedure to use these results in the calculation of vulnerability functions (i.e. the probability distribution of loss for a set of intensity measure levels) that propagates the uncertainties from the fragility functions and consequence functions (which relate damage to loss) is also presented. All of these efforts have been developed within an open-source and transparent philosophy and therefore, all of these calculators can be found in a public code repository at GitHub [51].

2.1. Summary of DBELA

The DBELA methodology is a simplified nonlinear static analysis method for the seismic risk assessment of buildings. The method builds upon the urban assessment methodology proposed by Calvi [10], in which the principles of structural mechanics and seismic response of buildings were used to estimate the seismic vulnerability of classes of buildings. In this method, the displacement capacity and demand for a number of limit states needs to be calculated. Each limit state marks the threshold between the levels of damage that a building might withstand, usually described by a reduction in strength or by exceedance of certain displacement/drift levels. Once these parameters are obtained, the displacement capacity of the first limit state is compared with the respective demand. If the demand exceeds the capacity, the next limit states need to be checked successively, until the demand no longer exceeds the capacity and the building damage state can be defined. If the demand also exceeds the capacity of the last limit state, the building is assumed to have collapsed. This procedure is schematically depicted in Fig. 1, in which the capacities for three limit states are represented by Δ_i and the associated demand by Sd_i.

In this example, the demand exceeds the capacity in the first and second limit state but not in the third limit state, thus allocating the building to the third damage state.

2.1.1. Displacement capacity

As explained above, the demand in this methodology is represented by a displacement spectrum which can be described as providing the expected displacement induced by an earthquake on a single-degree-of-freedom (SDOF) oscillator of given period and damping. Therefore, the displacement capacity equations that are derived must describe the capacity of a SDOF substitute/equivalent structure and hence must give the displacement capacity at a given limit state (which could be structural or non-structural) at the centre of seismic force of the original structure.

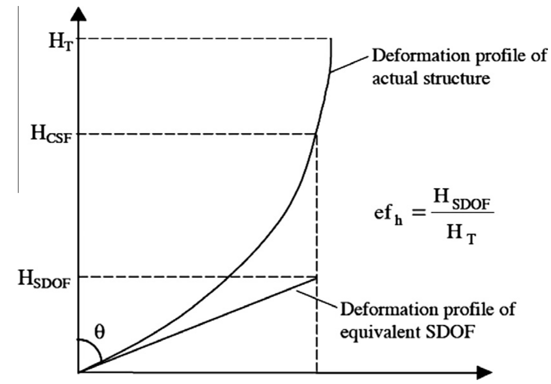

Fig. 2. Definition of effective height coefficient [24].

When considering structural limit states, the displacement at the height of the centre of seismic force of the original structure (H_{CSF}) can be estimated by multiplying the base rotation by the height of the equivalent SDOF structure (H_{SDOF}), which is obtained by multiplying the total height of the actual structure (H_T) by an effective height ratio (ef_h) (see Fig. 2).

Pinho et al. [37] and Glaister and Pinho [24] proposed formulae for estimating the effective height coefficient for different response mechanisms. For what concerns the beam sway mechanism (or distributed plasticity mechanism, as shown in Fig. 3), a ratio of 0.64 is proposed for structures with 4 or less storeys, and 0.44 for structures with 20 or more storeys. For any structures that might fall within these limits, linear interpolation should be employed. With regards to the column-sway mechanism (or concentrated plasticity mechanism, as shown in Fig. 3), the deformed shapes vary from a linear profile (pre-yield) to a nonlinear profile (post-yield). As described in Glaister and Pinho [24], a coefficient of 0.67 is assumed for the pre-yield response and the following simplified formula can be applied post-yield (to attempt to account for the ductility dependence of the effective height post-yield coefficient):

$$ef_h = 0.67 - 0.17\frac{\varepsilon_{s(LS_i)-\varepsilon_y}}{\varepsilon_{s(LS_i)}} \qquad (1)$$

The displacement capacity at different limit states (either at yield (Δ_y) or post-yield (Δ_{LSi})) for bare frame structures can be computed using simplified formulae, which are distinct if the structure is

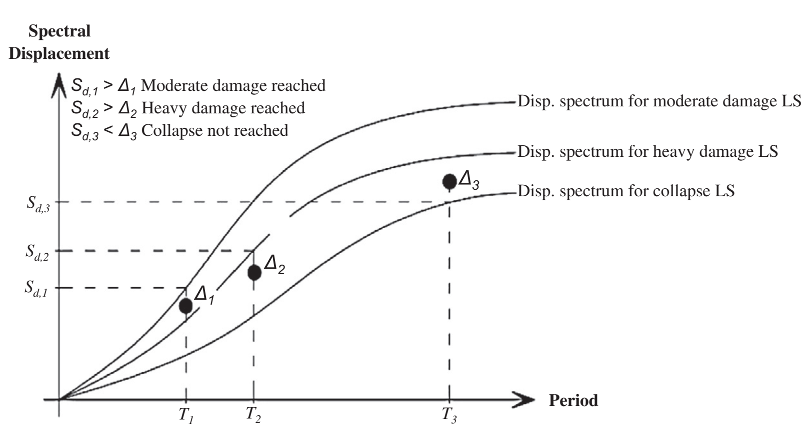

Fig. 1. Comparison between limit state capacity and the associated demand (adapted from [7]).