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Advances in the derivation of fragility functions for the development of risk-targeted hazard maps

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ABSTRACT

Risk-targeted hazard assessment aims at estimating the design ground shaking that leads to a uniform distribution of the collapse probability of buildings within a given region. An essential aspect of this methodology relies on the definition of the relationship between the collapse probability of buildings designed according to modern seismic regulations and the considered design ground motion. This study adds to previous research on the topic of seismic risk-targeted hazard assessment by investigating how the collapse probability varies with the design level of ground motion, and how this variability influences the resulting seismic risk across Europe. A large number of structures designed according to the most recent seismic regulation in Europe have been analysed. These structures were designed for increasing levels of peak ground acceleration ranging from values close to zero (i.e. buildings located in regions with very low seismic hazard) up to 0.40 g (i.e. moderate to high seismic hazard). Each structure was modelled as a tri-dimensional finite element model, and tested against a set of ground motion records using nonlinear dynamic analyses. Several fragility functions were derived for yielding and collapse damage states, and combined with the seismic hazard curves from the European project SHARE to calculate the spatial distribution of earthquake risk across Europe.

1. Introduction

Current seismic design codes require buildings to be designed for a given ground motion intensity level that is determined from a prescribed return period. Often the implicit assumption is that this design criterion ensures that the probability of collapse of different types of buildings, although often unknown, is comparable if not uniform. This is unfortunately not true. Uncertainties in the fragility of structures and on the shape of the local hazard curves often lead to an associated seismic risk level that is not only site-specific but also structure-specific, thus invalidating the previous hypothesis [20,10].

Luco et al. [20] has described a methodology, known as risk-targeted hazard assessment, that aims at computing the ground motion intensity that in fact leads to an uniform distribution of the seismic risk within a region, usually at national level. The target seismic risk level is directly correlated to the risk a given community or society is willing to accept, and should be established by decision makers, such as politicians with the support of engineers and sociologists.

In this context, it is clear that following a design methodology based on the principles of risk-target assessment has several advantages in comparison with the current procedures. However, a significant obstacle in the implementation of this methodology still needs to be overcome. An essential aspect of this methodology relies on the relationship between the design ground motion (a_{des}) and the expected collapse probability of the structure given a ground motion intensity ($P_c|a_g$). Low values for the probability of collapse at the design ground motion are to be expected for newly designed structures. However, a literature review has revealed an extremely high variability in this parameter, ranging from 10^{-7} to 10^{-1} [20,10,27,33,21,31]. The highest value has been reported for the United States, but it should be noted that this probability of collapse is for a design ground motion for a longer return period (i.e. maximum considered earthquake (MCE) at 2475 years). These studies, however, considered different types of buildings and design regulations, which often required the use of values of a_{des} corresponding to distinct return periods. Appropriate boundaries for $P_c|a_{des}$ and its associated dispersion can be defined by analysing large suits of structures designed according to the same criteria. The ground motion intensity, a_g, at which the collapse is reached is usually

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modelled as a random variable with a cumulative lognormal distribution, defined by a logarithmic standard deviation (β) and by any quantile of the distribution (e.g. the 50th quantile, namely the median, $\hat{a}_c$ for which $P_c|\hat{a}_c = 0.5$). High dispersion for β has also been found with proposed values ranging from 0.4 to 1.0 [28]. This parameter has a significant influence on the resulting risk-targeted hazard results, as it affects directly the slope of the fragility curve. For example, although not often recognized, large values of β in the widely adopted lognormal distribution modelling framework cause non-negligible values of $P_c|a_g$ for levels of ground motion that are hardly felt by humans and certainly of no harm to engineered structures such as those considered here. This large variability and its direct impact in the resulting earthquake risk strengthens the need to further investigate reasonable ranges for $P_c|a_{des}$ and β.

Despite the obvious need for providing an adequate safety margin against collapse when designing and constructing new structures, it is also important to minimize the potential losses due to extensive damage for more frequent events. Observations of past events have revealed regions where modern seismic design regulations are well established, but still high economic losses have been reported. For example, the 1994 Northridge earthquake is deemed as one of the costliest seismic events in recent history, and most of the economic losses came from severely damaged structures, and not due to the very limited number of collapses. These considerations have already been accounted for in some design regulations, such as the Eurocode 8 [8], which establishes a damage limitation requirement for a design ground motion corresponding to a probability of exceedance of 50% in 50 years. However, such an approach, once again, leads to an uneven distribution of damage risk across different structures and regions.

This study investigates the structural fragility of new buildings designed according to the European regulation, within the context of risk-targeted hazard assessment. This goal is achieved through numerical modelling of a number of structures designed considering different seismic ground motion hazard levels, which are then utilized to perform numerous nonlinear dynamic analyses (NDA). The building responses resulting from the NDAs are combined with a damage model to derive fragility functions for yielding (representing the onset of damage) and structural collapse. A comparison is also made between existing fragility functions and those developed herein. Conclusions are drawn regarding the impact that fragility curves with different characteristics have on the annual probability of collapse or of reaching structural damage for different buildings across Europe. Moreover, the findings presented herein also allow estimating fragility curves for any region in Europe, provided that the buildings have been designed according to the Eurocode.

2. Numerical modelling and ground motion selection

2.1. Structural design and 3D finite element modelling

For this study a pre and post-processing Matlab® [22] algorithm has been developed and all the structural analyses have been performed with the open-source finite element software OpenSees [23].

The case study buildings are reinforced concrete moment frames designed according to the most up-to-date European regulations [5–8]. All the structures are regular in height and symmetric along both horizontal main axes. The concrete class chosen for the structural design has a characteristic strength of 25 MPa, whilst the characteristic yield stress of the rebar steel considered herein was 500 MPa. A permanent load of 6.25 kN m^{-2} has been considered on all floors to reproduce the weight of a reinforced concrete slab of average thickness. Following the guidelines of Eurocode 1-1 [5] for residential buildings, an additional live load of 2.80 kN m^{-2} has also been considered in the design stage. For the top floor (roof) the absolute value of the live load has been lowered to 0.40 kN m^{-2}. In addition to the vertical loads, all structures have been designed to withstand horizontal loading due to the wind excitation, considering a wind velocity of 25 ms^{-1} and a Class II terrain, according to the Eurocode 1-4 [6].

To avoid excessive deformations under static loading, all beams have been designed with a minimum height equal to 1/12 of the span length, while the minimum cross section considered for columns was $0.25 \times 0.25\,\mathrm{m}^2$. Standard values for the reinforcing bars diameters have been used (i.e. 6, 8, 10, 12, 20, 25 and 32 mm). The adopted rebar pattern was the one that minimized the difference between the required rebar area and the actual rebar area while ensuring sufficient spacing between rebars in the most congested cross sections. When designing the structural elements, if the cross section of any structural components had to be updated, the minimum increment in the section's dimensions considered was 0.05 m.

Five sets of 10 structures with 3 and 5 storeys designed for 5 increasing levels of ground motion ranging from peak ground acceleration (PGA) of 0.05 g (i.e. very low seismic hazard) up to 0.4 g (i.e. moderate to high seismic hazard) have been analysed herein. Eurocode 8 [8] performance requirements and recommendations have been followed during the design phase. At first, the selected number of storeys might seem quite similar (i.e. 3 *vs.* 5 storeys), however it should be noted that these two sets of structures have considerably different dynamic properties (e.g. differences in the mean period in Fig. 2-top and -bottom). These differences are significant enough to influence the design process with, for example, not being uncommon for the period of vibration of the 3-storeys frames to land in the plateau interval of the design spectrum while for the 5-storey frames the design spectral acceleration is computed from the descending branch of the spectrum.

In order to introduce variability in the design, the span length and storey height have been randomly sampled from the probability

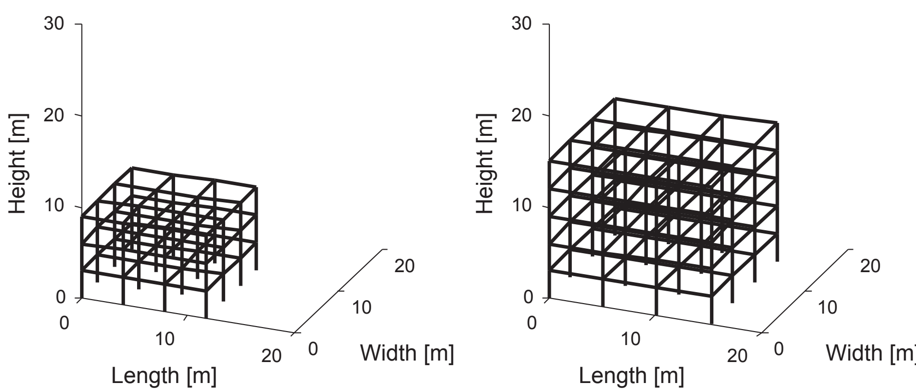

Fig. 1. Representation of the numerical models (Left) 3 storeys building; (Right) 5 storeys building.