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A closed form solution for seismic risk assessment incorporating intensity bounds

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ABSTRACT

A closed-form solution of the risk equation incorporating intensity bounds is derived and analysed. The new equation, compared to the well-known risk equation developed in the 1990s, includes a correction factor, which has a value less than one if the effect of the intensity bounds is significant. The lower bound of ground-motion intensity represents a minimum ground-motion intensity, which causes a designated limit state, whereas the upper bound of ground-motion intensity is, in general, related to the physics of earthquakes, the tectonic regime, and the geology of the terrain in the region from the epicentre to the site of the building. In the paper typical values of the minimum collapse intensity and of the fragility parameters of code-conforming frames are discussed. An approximate procedure for assessing the upper bound of ground-motion intensity on the basis of ground-motion prediction models is also proposed. Finally, the procedure for seismic risk assessment is demonstrated by assessing the collapse risk for a 4-storey and a 15-storey building. It is shown that the collapse risk assessed on the basis of peak ground acceleration can be significantly affected by the lower bound of the collapse intensity, whereas the impact of the upper bound of the ground-motion intensity on the collapse risk can be more pronounced when the assessment of the collapse risk is based on the spectral acceleration at the first vibration period.

1. Introduction

The procedures for the design of structures which are currently prescribed in structural codes and used in practice are far from the performance-based earthquake engineering concept. In Europe, seismic design is based on a linear elastic method of analysis and a so-called design earthquake, which is often related to a mean return period of 475 years. For this reason current standards for the earthquake-resistant design of structures do not guarantee a tolerable collapse risk for all structures, which is the most important objective in the case of performance-based earthquake engineering. On the other hand, seismologists are focused on providing hazard maps for relatively short return periods, which do not have significant impact on the collapse risk. However, recently risk-targeting hazard maps have been introduced [1,2], which partly solve the issue of this obsolete practice for the definition of design earthquakes. In a more general case, at least the important structures should be designed utilising the risk-based seismic design procedure [3], which takes into account comprehensive information regarding seismic hazard and ensures a target reliability of structures. Risk-based seismic design is an iterative procedure, which involves seismic performance assessment based on nonlinear analysis methods, seismic risk assessment, which is in the simplest case expressed by the mean annual frequency of limit-state exceedance, and structural adjustment.

Since seismic risk assessment is steadily becoming a part of the seismic design of structures, it is important to understand which parameters have the greatest impact on seismic risk. For this purpose, it makes sense to use the closed-form solution of the risk equation. The first variant of this equation was developed in 1990 during discussions between G.R. Toro and C.A. Cornell, as noted by McGuire [4], and used for different purposes [5–9]. Based on the closed-form solution of the risk equation [4,10] it is clear that seismic risk is primarily affected by the limit-state fragility parameters and by the slope of the hazard curve in log–log coordinates. However, it has recently been argued that the integration of the risk equation over the entire range of ground-motion intensity, e.g. from zero to infinity, is unphysical, since lower and upper bounds of intensity exist [11], respectively, due to the nature of a structure and the ground motions, and the seismicity in the area close to the site of the structure. Similarly, the risk equation is integrated from zero to an infinite intensity in the process of computing the risk-targeting hazard maps [1,2,12]. Such an approach in general overestimates the seismic risk. The question arises as to when the

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Nomenclature

LS	limit state
C	collapse limit state
im	intensity measure
im_{LS}	limit-state intensity
$im_{LS,50}$	median limit-state intensity
im_1	lower bound of the ground-motion intensity, i.e. a minimum intensity, which causes a designated limit state
im_2	upper bound of the ground-motion intensity
Δim_1	ratio between im_1 and $im_{LS,50}$
Δim_2	ratio between im_2 and $im_{LS,50}$
$\beta_{im,LS}$	standard deviation of the natural logarithms of limit-state intensities
λ_{LS}	mean annual frequency of limit-state exceedance
$\lambda_{LS,im1}$	λ_{LS} incorporating im_1
$\lambda_{LS,im12}$	λ_{LS} incorporating im_1 and im_2
C_f	correction factor due to the variability of the limit-state intensity
C_1	correction factor due to im_1
C_{12}	correction factor due to im_1 and im_2
$H(im)$	seismic hazard function
k	slope of the hazard function in log–log coordinates
k_0	annual rate of exceedance of $im = 1$
a_g	peak ground acceleration
$a_{g,R}$	design peak ground acceleration at rock outcrop
$a_{g,d}$	design peak ground acceleration
$a_{g,C}$	peak ground acceleration causing collapse
$a_{g,C,50}$	median peak ground acceleration causing collapse
$a_{g,1}$	lower bound of the peak ground acceleration
$a_{g,2}$	upper bound of the peak ground acceleration
$\Delta a_{g,1}$	ratio between $a_{g,1}$ and $a_{g,C,50}$
$\beta_{ag,C}$	standard deviation of the natural logarithms of the peak ground accelerations causing collapse
$S_a(T_1)$	spectral acceleration at the first vibration period
T_1	first vibration period of the structure
$S_{a,d}$	design spectral acceleration at the first vibration period
$S_{a,C}$	spectral acceleration at the first vibration period causing collapse
$S_{a,C,50}$	median spectral acceleration at the first vibration period causing collapse
$S_{a,1}$	lower bound of spectral acceleration at the first vibration period
$S_{a,2}$	upper bound of spectral acceleration at the first vibration period
$\Delta S_{a,1}$	ratio between $S_{a,1}$ and $S_{a,C,50}$
$\beta_{Sa,C}$	standard deviation of the natural logarithms of spectral accelerations at the first vibration causing collapse
M	magnitude
R_{jb}	Joyner–Boore distance
σ	standard deviation of the predicted ground-motion intensity
$\tilde{a}_g$	median predicted peak ground acceleration
$a_{g,2,2\sigma}$	predicted upper bound of the peak ground acceleration at 2σ above the median
$a_{g,2,3\sigma}$	predicted upper bound of the peak ground acceleration at 3σ above the median
$a_{g,2,\min}$	minimum predicted upper bound of the peak ground acceleration
$a_{g,2,\max}$	maximum predicted upper bound of the peak ground acceleration
$\tilde{S}_a$	median predicted spectral acceleration
$S_{a,2,2\sigma}$	predicted upper bound of the spectral acceleration at 2σ above the median
$S_{a,2,3\sigma}$	predicted upper bound of the spectral acceleration at 3σ above the median
$S_{a,2,\min}$	minimum predicted upper bound of the spectral acceleration
$S_{a,2,\max}$	maximum predicted upper bound of the spectral acceleration

overestimation of seismic risk becomes significant if the intensity bounds are not selected in a physically consistent manner.

In order to answer this question a closed-form solution of the risk equation incorporating intensity bounds is firstly derived in this paper. An insight into the parameters which have the greatest impact on the assessment of seismic risk is then achieved by analysing the new closed-form equation, which, if compared to the well-known equation from the 1990s, differs by a correction factor which incorporates the effect of the lower and upper bounds of the ground-motion intensity. A discussion on the assessment of fragility parameters and the minimum collapse intensity of code-conforming frames follows, together with a discussion on an approximate procedure for the assessment of the upper bound of ground-motion intensity, which may not necessarily be understood as the physical limit of intensity. The impact of the intensity bounds on the assessment of collapse risk is then demonstrated for a 4-storey and a 15-storey frame building by utilising the peak ground acceleration and the spectral acceleration at the first vibration period as intensity measures. It should be emphasised that the closed form solution of the risk equation is different to that previously published before [11]. In this case the solution is based on the truncated limit-state fragility function, which takes into account the fact that the probability of exceeding a designated limit state is zero if the ground-motion intensity is lower than the minimum ground-motion intensity which causes a designated limit state (e.g. collapse). Thus the new closed-form solution of the risk equation can be used to assess, for example, the probability of collapse with the consideration of the intensity bounds.

2. Closed-form solution of the risk equation incorporating intensity bounds

2.1. Theoretical background and derivations

The mean annual frequency (MAF) of limit-state exceedance is defined by the following integral (e.g. [13–17]):

$$\lambda_{LS} = \int_0^{\infty} P(LS|IM = im) \cdot \left|\frac{dH(im)}{d(im)}\right| \cdot d(im)\,, \tag{1}$$

where the fragility function $P(LS|IM = im)$ is the probability of exceeding the limit state (LS) if the intensity measure (IM) takes on a value equal to im and the hazard curve $H(im)$ is the annual rate of exceedance of im. The fragility function can be simply defined as the cumulative distribution function of the limit-state intensity $P(IM_{LS} < im)$. If it is assumed that the limit-state intensity is log-normally distributed, the fragility function can be expressed by means of the standard normal probability integral [10]:

$$P(IM_{LS} < im) \approx \Phi\left[\frac{\ln(im) - \ln(im_{LS,50})}{\beta_{im,LS}}\right], \tag{2}$$

where $im_{LS,50}$ and $\beta_{im,LS}$ are the median limit-state intensity and the corresponding standard deviation of the natural logarithms and $\Phi(\cdot)$ is the cumulative distribution function of the standard normal variable. If it is additionally assumed that the hazard curve is linear in log–log coordinates:

$$H(im) = k_0 \cdot im^{-k}, \tag{3}$$