ASSOCIAZIONE NAZIONALE ITALIANA DI INGEGNERIA SISMICA ANIDIS
Italian National Association of Earthquake Engineering

XVIII CONVEGNO ANIDIS
L'ingegneria sismica in Italia
ASCOLI PICENO 2019
15-19 Settembre

Risk assessment of bridge's piers subjected to multiple earthquakes

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Keywords: Multiple earthquake, seismic damage index, accumulation process, exceeding probability, lifetime, bridge piers.

ABSTRACT

Structures and infrastructures located in seismic regions are continuously exposed to the earthquake hazard and during their design lifetime may be subjected to multiple earthquakes, including multiple main shocks or main shock-aftershock sequences. Different probabilistic methodologies have been recently proposed to evaluate the performance of structures under repeated earthquake sequences of different intensity. These include the method based on a Markovian modelling of seismic damage accumulation, and the method based on a probabilistic seismic demand model under multiple earthquake events. These methods are applied to evaluate the probabilistic damage accumulation of a reinforced concrete bridge pier modelled in OPENSEES, under a seismic scenario based on the Atkinson-Silva stochastic ground motion model. The obtained results are compared to those obtained by applying a pure Montecarlo approach to evaluate the accuracy and efficiency of the proposed methods.

1 INTRODUCTION

Structures and infrastructures are usually located in regions subjected to repeated seismic excitations during their design life. Multiple earthquakes can, in a long period, lead to a reduction in structural capacity and ultimately to the collapse of the construction with a devastating impact on the urban context in terms of human lives and economic losses. So, considering the prospect of potential future destructive events, the seismic risk assessment has to be focused on probabilistic models able to best predict future scenarios and consider the high uncertainties involved in the analysis (Castaldo et al., 2018). As known, within the performance-based seismic framework (PBEE) (Krawinkler, 1999), the seismic risk represents the probability of losses occurring due to earthquakes, in a fame time period, in terms of human lives, social disruption as well as economic losses. Moreover, it is a function of the site seismic hazard, computed though the probabilistic seismic hazard analysis (PSHA) (Iervolino et al., 2013), of the seismic vulnerability of the structural systems and of the exposure. In this context, innovating methodologies aimed at evaluating the progressive structural degradation due to the accumulation of damage for repeated main-shock events during the lifetime of structures have been recently proposed. Specifically, Ghosh et al. (2015) proposed an approach based on predictive regression models for statistically predicting damage accumulation based on earthquake intensity and damage history. These models are also used to predict the probability of damage index exceedance conditioned on the number of earthquake pulses incurred by the structure. Using generated ground motions, several nonlinear incremental dynamic analyses of a reinforced concrete (RC) bridge pier are conducted to evaluate the relevant response parameters assuming appropriate local and global seismic damage indices for the performance assessment in probabilistic terms. The results demonstrate a significant increase in the damage index exceedance probabilities due to repeated main-shocks during the lifetime confirming that the accumulation process cannot be neglected.

2 FRAMEWORK OF THE DAMAGE ACCUMULATION FORMULATION

Failure of a structural system subjected to seismic sequence may be due to the exceedance of the collapse limit state. The exceeding probability of a generic cumulative engineering demand parameter D with respect to a value d can be expressed through the total probability theorem as follows:

$$P\left[D \geq d\right] = \sum_{1}^{\infty} P\left[D \geq d \mid n\right] \cdot P\left[n, T\right] \tag{1}$$

where $P\left[D \geq d \mid n\right]$ is the probability that the demand D exceeds d, conditional to have n shocks, and $P\left[n, T\right]$ is the probability of having n shocks in the design lifetime T. The latter quantity can be expressed by mean of a homogeneous or non-homogeneous Poisson assumption to model the occurrence of the main shock or aftershock events. The present study investigates the evolution of engineering demand parameters, discussed in the next sub-sections, for the main-shock scenarios employing Ghosh et al. method for evaluating $P\left[D \geq d \middle| n\right]$.

2.1 *Engineering demand parameters (EDPs)*

In line with the PBEE (Kumar, 2012), in this work, the structural response is described using both global (i.e., Park and Ang) and local engineering demand parameters (EDPs) that can be used to predict the seismic damage to structural and non-structural components (Park et al., 1985). The local EDPs herein employed are: the maximum strain of confined (core) and unconfined (cover) concrete under compression ε_{ccore} and ε_{ccov}, the maximum strain of unconfined concrete in tension ε_{tcov} and the maximum strain of steel reinforcement under compression ε_{cs}. As an example in the following sections are shown the results of ε_{ccore}.

2.2 *Ghosh et al. method (2015)*

The approach proposed by (Ghosh et al., 2015) focuses on the assessment of damage accumulation under repeated shocks and is based on a probabilistic regression model taking into account both mainshocks and aftershocks. Only the first scenario corresponding to mainshocks has been examined in this study. According to (Cornell et al., 2005), for a single shock event, the relationship between the median structural demand, EDP, and the *IM* (intensity measure) can be approximated as follows:

$$EDP = a(IM)^b \tag{2}$$

where a and b are the regression coefficients. For structure with nonlinear behaviour, the linear regression model in the log-log space could be not valid for the entire *IM* range of interest. In fact, it has been found by (Tubaldi et al., 2016) that a good fit of local EDPs, such as material stress and strain, can be obtained by adopting a bilinear regression. It follows that the regression model is described by the following expression:

$$\begin{aligned} \ln EDP_1 \mid IM &= (a + b \ln IM_1) H + \\ &+ (c + d \ln IM_1)(1 - H) + \ln \varepsilon \left| IM \right. \end{aligned} \tag{3}$$

where a, b, c and d are the regression coefficients related to the slope of the two linear segments, $\ln \varepsilon \left| IM \right.$ is the error variable relative to the regression and H is the step function that is $H = 1$ for $IM \leq IM^*$, $H = 0$ for $IM \geq IM^*$. The IM^* parameter identifies the breakpoint, which is defined as the intersection point of the two linear segments. The unknown coefficients, IM^* and $\ln \varepsilon \left| IM \right.$ can be evaluated through a nonlinear least square regression. The evaluation of the cumulative damage due to multiple earthquakes is further characterised by its dependence on the history of seismic events. In compliance with (Ghosh et al., 2015) the damage index after n earthquake shocks can be described as a multi-linear regression model as follows:

$$\begin{aligned} \ln EDP_n \left| IM, \mathrm{EDP}_{n-1} \right. &= (a_n + b_n \ln EDP_{n-1}) H_n + \\ &+ (c_n \ln IM_n + d_n \ln IM_n \ln EDP_{n-1})(1 - H_n) + \\ &+ \ln \varepsilon \left| IM_n, EDP_{n-1} \right. \end{aligned} \tag{4}$$

where EDP_n is the damage index after the n-th earthquake shock of a sequence with the respective ground motion intensity IM_n; a_n, b_n, c_n and d_n are the regression coefficients related to the slope of the two surfaces, H_n is the step function that is $H_n = 1$ for $IM \leq IM^*$ and $H_n = 0$ for $IM > IM^*$ and $\ln \varepsilon \left| IM_n, EDP_{n-1} \right.$ is the error variable relative to regression. This multi-linear regression model can be seen as an extension of the model presented in Eq.(3) because the damage index of the structure after the n-th shock of a sequence depends on how "weak" the structure has become after being exposed to the previous (n - 1) shocks (quantified by EDP_{n-1}). In order to apply the described probabilistic demand model, the structure must be subjected to a series of several ground motions (s) in the form of a train of several earthquakes in order to represent the occurrence of seismic shocks in the design life of the structure. Once the regressive model is applied, the probability exceeding the damage levels can be assessed taking into account that the structure is subjected to a certain number of shocks. Monte Carlo simulation is used to generate several earthquake intensity measures (*IMs*), sampled on the basis of the site-specific seismic hazard curves. The predictive equations are then applied based on the data deriving from the nonlinear dynamic analyses of the structure subjected to the seismic sequences. At last, the probability exceeding a threshold damage level is estimated as follows:

$$P\left[D \geq d\right] = \frac{1}{N}\sum_{i} I\left[D_{ni} > d\right] \tag{5}$$

with N is the total number of Monte Carlo samples, D_{ni} represents the damage index after n shocks for the i-th Monte Carlo sample, and $I[\bullet]$is a step function equals to 1 when $I[\bullet]$ is true or to 0 if not. According to (Ghosh et al., 2015), this study employs 50000 Monte Carlo trials (N) sampled from the regression model to define accurate estimates of the damage index exceedance probabilities.

3 STRUCTURAL MODELS: GEOMETRY AND MATERIALS

The selected case study is a RC bridge pier of denoted as 815 (Lehman et al., 2000). The details of the considered bridge pier are summarised in Table 2. As shown in Table 2, the piers is characterized by cross-section with diameter D_m equal to 610 mm, a slenderness ratio L/D_m (Figure 1). Nonlinear behaviour of the RC column has been modelled in OpenSees by means of the fibre-based section discretisation technique (Berry et al., 2006) where the behaviour of the pier is modelled using a beam-column element and the cross-section of the element is discretized into a number of steel and concrete fibres at the selected integration points. This research employs the models in (Kashani et al., 2016) in which three integration points using Gauss–Lobatto integration scheme are employed, based on the recommendations provided by (Coleman et al., 2001); while a force-based element with five integration points is considered to model the top part of the column (Berry et al., 2006). A schematic view of the fibre model and fibre sections is shown in Figure 2. The column has the following mechanical properties: steel yield stress f_y=540 MPa, maximum deformations of confided and unconfined concrete under compression and tension, respectively, are ε_{ccore}=0.035, ε_{ccover}=0.00428, ε_{tcover}=0.00125, while the maximum deformation of the steel under compression is ε_{cs}=0.08.

Table 1. Details of the structural models.

Col ID	Length L (mm)	L/D_m	Vertical Φ (mm)	Horizontal Φ (mm)
815	4876.8	8	16	6.5

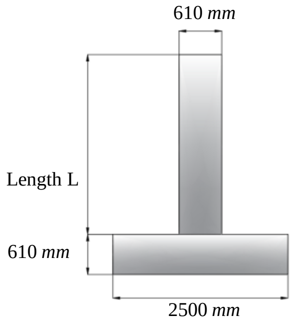

Figure 1. Schematic view of the bridge piers.

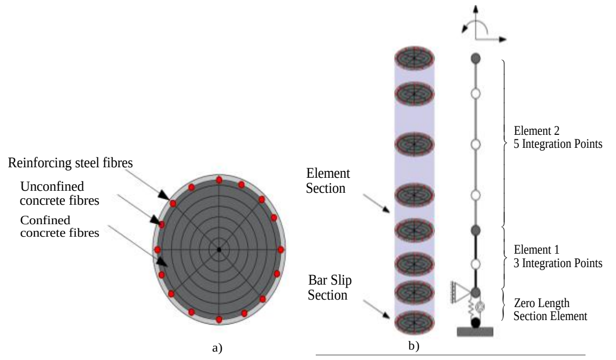

Figure 2. Schematization of fibre beam-column element (a) with the bar buckling and bar slip model (b).

The material nonlinearity is described through a uniaxial material relationship for steel (tension and compression) and concrete (confined and unconfined). Concrete04 available in OpenSees (Pugh, 2012) is used for the analyses to model the unconfined concrete behaviour in cover concrete and the confined concrete in the core of the columns restricted by the reinforcement. This model is based on the curve of (Kashani et al., 2016) in the compression and a linear-exponential decay curve in tension. To account for the stiffness degradation and determine the unloading-reloading stiffness in compression, the Karsan–Jirsa model is used. The secant stiffness is used to define the unloading/reloading stiffness in tension. The confinement parameters are taken from (Karsan ID) To describe the behaviour of steel reinforcement the phenomenological uniaxial model developed by (Mander et al., 1988) has been used. For its implementation the hysteretic material model available in OpenSees (OpenSeee, 2011) was used, then the generic uniaxial fatigue material developed by (Mander et al., 1988) has been wrapped to the previous one, in order to simulate the slow cycle fatigue failure of vertical reinforcing bars. It is a combined model that takes into account the influence of inelastic buckling and low cycle fatigue degradation jointly. Further details on the considered model are available in (Karsan et al., 1969).

4 GROUND MOTIONS GENERATION

The uncertainties in the seismic input are taken into account by means of a fully stochastic method (Dall'Asta et al., 2017). This technique is the Atkinson and Silva ground motion (GMAS) model (Kashani et al., 2016) in which the *IM* is achieved for a seismic source by defining the moment magnitude M_m, the hypo-central distance R together with the ground motion radiation spectrum A(f) and the time envelope function e(t). Because of the uncertainty related to event location, the GMAS assumes that earthquakes of magnitude between M_{min} and M_{max} equally occur likely in a circular area of radius R_{max} centred at the site where the structure is situated. In Table 3 the input data for our pier are reported: R_{max} is the maximum hypo-central distance, M_{max} and M_{min} are the maximum and minimum magnitude, respectively, T_{pier} is the period of the considered structure and ν_{eq} the mean annual frequency (MAF) of earthquakes of any significant intensity. Through the GMAS method 5000 accelerograms have been generated. Then, through the Monte Carlo method an increasing number (i.e., s = from 200 to 3500) of ground motions has randomly been extracted from those previously generated. The result is a number of multiple mainshocks, consisting of 20 occurrences, that represent the range of the seismic events that can occur in the design period of the structure and have to be applied to the pier in order to evaluate the EPDs of interest. So, the selected bridge pier has been subjected to seismic sequences, in first instance, considering the occurrence of twenty shocks over their lifetime. Moreover, a further study has also been conducted to assess the maximum number of events to be considered in the design life T=50 years. It has been seen that, by increasing the number of occurrences, the probability of collapse increases significantly, reaching, after about ten earthquakes, the 100% probability of exceeding the first level of damage.

Table 2. Input data for ground motion generation.

ID	R_{max} [Km]	M_{max}	M_{min}	T_{pier} [s]	ν_{eq}
815	10	8	5.5	0.69	0.0997

5 RISK ASSESSMENT RESULTS

This section describes the results from the dynamic analyses of the bridge pier subjected to multiple earthquakes (i.e., s=200-3500), as a seismic train composed of 10 pre-arranged shocks, randomly extracted through Monte Carlo simulation from the database of 5000 generated ground motions. Once the responses of the structures have been obtained, the risk assessment has been performed through Ghosh et al. (2015) approach. Figure 3 shows the curves resulting from the application of the method explained in Section 3.

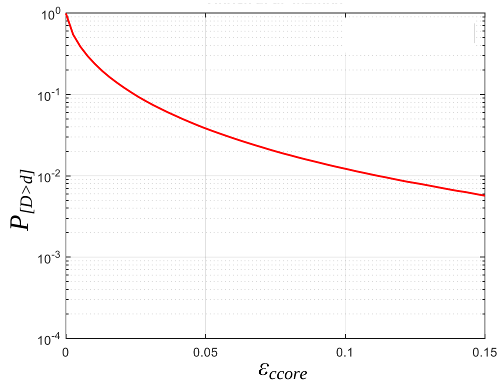

Figure 3. Column 815: exceedance probability for ε_{ccore}.

The results demonstrate a significant increase in the damage index exceedance probabilities due to repeated main-shocks during the lifetime confirming that the accumulation process cannot be neglected.

6 CONCLUSIONS

Using generated ground motions, several nonlinear incremental dynamic analyses of a RC bridge pier are conducted to evaluate the relevant response parameters assuming appropriate local and global seismic damage indices for the performance assessment in probabilistic terms. The results demonstrate a significant increase in the damage index exceedance probabilities due to repeated main-shocks during the lifetime confirming that the accumulation process cannot be neglected.

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