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Incremental dynamic analysis applied to seismic financial risk assessment of bridges

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Abstract

Incremental dynamic analysis (IDA) is applied in a performance-based earthquake engineering context to investigate expected structural response, damage outcomes, and financial loss from highway bridges. This quantitative risk analysis procedure consists of: adopting a suitable suite of ground motions and performing IDA on a nonlinear model of the prototype structure; summarizing and parameterizing the IDA results into various percentile performance bounds; and integrating the results with respect to hazard intensity–recurrence relations into a probabilistic risk format. An illustrative example of the procedure is given for reinforced concrete highway bridge piers, designed to New Zealand, Japan and Caltrans specifications. It is shown that bridges designed to a "Design Basis Earthquake" that has a 10% probability in 50 years with PGA $= 0.4g$, and detailed according to the specification of each country, should perform well without extensive damage. However, if a larger earthquake occurs, such as a maximum considered event which has a probability of 2% in 50 years, then extensive damage with the possibility of collapse may be expected. The financial implications of this vulnerability are also given, revealing a fourfold variation between the three countries.

Keywords: Incremental dynamic analysis; Performance based earthquake engineering (PBEE); Expected annual loss; Seismic risk assessment

1. Introduction

Performance based earthquake engineering procedures require the prediction of the seismic capacity of structures which is then compared to the local seismic demand. The interrelationship between the two gives an inference of the expected level of damage for a given level of ground shaking. In order to estimate structural performance under seismic loads, Vamvatsikos and Cornell [1] proposed a computational-based methodology called *incremental dynamic analysis* (IDA). The IDA approach is a new methodology which can give a clear indication of the relationship between the seismic capacity and the demand. With respect to seismological *intensity measures* (IM), such as *peak ground acceleration* (PGA), engineers can estimate principal response quantities in terms of governing *engineering demand parameters* (EDP), such as the maximum deflection or drift of the structure.

The IDA approach involves performing nonlinear dynamic analyses of a prototype structural system under a suite of ground motion records, each scaled to several IM levels designed to force the structure all the way from elastic response to final global dynamic instability (collapse). From IDA curves, limit states can be defined. The probability of exceeding a specified limit state for a given IM (e.g. PGA) can also be found. The final results of IDA are thus in a suitable format to be conveniently integrated with a conventional seismic hazard curve in order to calculate mean annual frequency of exceeding a certain damage limit-state capacity. This can be used to determine likely damage given a scenario earthquake event, or in a financial sense, the *expected annual loss* (EAL), incorporating the entire range of seismic scenarios.

This paper develops the IDA process specifically for bridge structures. What is new here is the way in which IDA results are quantitatively modelled and then integrated into a probabilistic risk analysis procedure whereby the seismic intensity–recurrence relationship (the seismic demand) is viewed with respect to the damage propensity of a specific bridge structure (structural capacity). Confidence intervals and damage outcomes for given hazard intensity levels, such as the *design basis earthquake* (DBE) or a *maximum*

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considered earthquake (MCE), can be evaluated. In addition, a methodology for assessing the financial seismic vulnerability of bridges is introduced. The procedure will be demonstrated using a comparative study of bridge piers designed to three different countries' standards.

2. IDA-based seismic risk assessment

2.1. Step 1: Select ground motion records and hazard–recurrence risk relation

In order to perform IDA, a suite of ground motion records are needed. In their previous study, Vamvatsikos and Cornell [1] used 20 ground motion records to analyse mid-rise buildings in order to provide sufficient accuracy of seismic demands. The same ground motions as used by Vamvatsikos and Cornell [1] were adopted for this study. These earthquakes have Richter magnitudes in the range of 6.5–6.9 with moderate epi-central distances mostly in the range of 16–32 km; all these ground motions were recorded on firm soil. Fig. 1(a) shows response spectra for each of the 20 earthquake ground motions scaled to the same IM that is a PGA of $0.4g$. A significant degree of variability is evident with respect to the median spectral curve. Fig. 1(a) also presents a plot of the lognormal standard deviation (β_D), sometimes referred to as the dispersion, across the spectrum. Due to the consistent and relatively low values of β_D for periods up to 1.6 s, it is evident that PGA serves (for this suite of earthquakes) as an appropriate IM.

It is necessary to define a relationship between an IM and annual frequency, f_a. This is commonly known as the hazard–recurrence relationship. By fitting a straight line through two known points in a log–log scale, it is possible to approximate the hazard–recurrence curve by the following equation [2]:

$$f_a(\mathrm{IM}) = k_o(\mathrm{IM})^{-k} \tag{1}$$

where k_o and k are empirical constants. According to the data specified in design codes, values of k for New Zealand, Japan and Caltrans designs were determined to be 3.00, 2.40 and 3.45, respectively. These curves are given in Fig. 1(b).

2.2. Step 2: Perform incremental dynamic analysis

Once the model and the ground motion records have been chosen, IDA is performed. Thus a nonlinear computational model of the prototype structural system should be developed. To start the analysis, the chosen earthquake records need to be scaled from a low IM to several higher IM levels until structural collapse occurs.

For each increment of IM, a nonlinear dynamic time history analysis is performed. Analyses are repeated for higher IMs until structural collapse occurs. Locating the maximum drift observed in an analysis gives one point in the IM vs. EDP (PGA vs. drift) domain. As shown in Fig. 1(c), connecting such points obtained from all the analyses using each earthquake record with different IMs gives the IDA curves for all earthquakes in the suite.

It may also be of interest to analyse the variability of the response outcomes for a given level of IM. The authors have found the IM vs. EDP data of the 20 IDA curves conform to a lognormal distribution. Example data distributions are given for the EDP values at a small IM (indicative of the initial slope of the IDA curves) and for the IM values at a large EDP (indicative of the collapse IM) in Fig. 2, which include a lognormal distribution fitted to the data. Best fit lognormal mean (the median) and lognormal standard deviations for these curves were found using least-squares analyses. Following the Kolmogorov–Smirnov goodness-of-fit test principles, two curves representing 10% and 90% probability are also shown in the figure. The data fit well within the 10% significance levels, thereby validating the use of the lognormal distribution.

2.3. Step 3: Model the percentile IDA curves

In their previous study, Vamvatsikos and Cornell [1] modelled their IDA curves by using multiple interpolation spline functions. It is considered that such an approximation is cumbersome and not particularly useful for subsequent risk analysis. Therefore, in this study several single functional relations were explored, and the *Ramberg–Osgood* (R–O) equation was adopted as the most suitable. The R–O equation can be written in the following two forms:

$$\frac{\mathrm{EDP}}{\mathrm{EDP}_c} = \frac{\mathrm{IM}}{\mathrm{IM}_c} + \left(\frac{\mathrm{IM}}{\mathrm{IM}_c}\right)^r = \frac{\mathrm{IM}}{K\cdot\mathrm{EDP}_c}\left(1 + \left|\frac{\mathrm{IM}}{\mathrm{IM}_c}\right|^{r-1}\right) \tag{2}$$

in which K = the initial slope of the IDA curve in the proportional range; IM_c = "critical" intensity measure that occurs at the onset of large EDPs that subsequently lead to collapse; $\mathrm{EDP}_c = \mathrm{IM}_c/K$ is the "critical" value of EDP; and r = constant.

First the median IDA curve is derived by interpolation from the actual IDA data. Next the R–O equation is fitted to this observed median IDA curve. During calibration of this process, it has been found that the value of r in the R–O relation, varies little across a suite of IDA curves. Thus a reasonable value of r is fixed and the other parameters K and IM_c, and their associated dispersions β_K and β_{IMc} found by conducting a combined least squares analysis on interpolated 10th, 50th and 90th percentile curves. Fig. 1(d) illustrates the fit between the actual IDA data points and the fitted R–O curve for one specific case, and Fig. 1(e) illustrates fitted continuous smooth IDA curves representing the 10th, 50th and 90th percentile response demand.

2.4. Step 4: Assign damage limit states

Once the three (10th, 50th and 90th percentile) lines have been generated, it is possible to determine the expected drift for an earthquake with a certain level of intensity. Emerging international best practice for seismic design is tending to adopt a dual level intensity approach: (i) a DBE represented by a 10%