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Probabilistic seismic demand model for regular bridges based on the modified Park-Ang damage index

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Abstract. In this paper, a modified damage model is established to solve the non-convergence problem existing in the Park-Ang damage index. The scatter of the modified damage index is significantly reduced compared to the original Park-Ang damage index. Then the finite element analysis models of 8 representative regular bridges samples were established with the main parameters variable such as the column height. The total 19120 increment dynamic analysis (IDA) curves were obtained by using the IDA method. These IDA curves are expressed in terms of one intensity measure (IM), i.e., the spectral acceleration at the fundamental period with 5% damping, and one engineering demand parameter (EDP), i.e., the modified Park-Ang damage index. At last, the probabilistic seismic demand model of regular bridges was established by using probability analysis based on the result of IDA curves. It is found that the seismic damage demand is reasonably standard beta distributed at given IM levels. The quadratic equation is suitable to establish the correlation between the mean value of the seismic damage demand and the IM. It is concluded that the established probabilistic seismic demand model is suitable for 3 different site conditions and can largely simplify the computational effort of seismic demand in the performance-based seismic design and assessment of regular bridges.

1. Introduction

Recent research efforts have focused on the development of performance based seismic design and assessment in the framework of probability for structures [1]. The Pacific Earthquake Engineering Research (PEER) Centre has developed a new-generation performance-based earthquake engineering (PBEE) framework based on the full probability theory, which involved four generalised variables: the Intensity Measure (IM), the Engineering Demand Parameter (EDP), the Damage Measure (DM), and the Decision Variable [2-4]. The proposed PBEE framework is widely accepted since its introduction and considered a sound basis for further development [5-7]. The probabilistic seismic demand model (PSDM) is a key aspect of implementing the PBEE framework.

A PSDM [7] is a conditional probability statement that expresses the probability that a bridge or a bridge component (i.e., columns, bearings) experiences a certain level of demand (D_r) for a given earthquake intensity measure (IM) level, as shown in the following equation:

$$\text{Prob}_{\text{demand exceedance}} = P[D_r \geq d | \text{IM}] \quad (1)$$

One of the most crucial issues in PSDM is the quantized assessment of potential structural damage. In order to make damage measurable and predictable, the concept of damage index provides a



convenient approach to quantify damage and predict the residual capacity of a certain member after an earthquake.

Many damage indices have been proposed to relate the EDP to the DM. According to the parameters selected for the damage identifications, it's possible to identify different damage index types, namely ductility DI, stiffness degradation DI, low-cycle fatigue DI, dissipated energy DI and combination DI [8]. Cosenza et al. (1993) [9] compared different damaged indexes and found out that DI defined by only one parameter seem to have remarkable uncertainties while DI based on ductility and energy are more reliable.

The most widely known cumulative DI was proposed by Park and Ang (1985) [10], which is widely used because of its foundation on extensive experimental results and its verification by many control examinations of damaged structures in several earthquakes. The proposed damage criterion includes both the maximum response and the effect of repeated cyclic loading and considers that a structure is weakened or damaged by a combination of stress reversals and high stress excursions. Although it is one of the most frequently used index, the Park-Ang damage index (DPA) provides insufficient real estimation of the damage for the limit values [11]. In order to solve the non-convergence problem, some modified Park-Ang indexes have been developed in the last years, i.e. Jiang et al. (2011) [11] developed a cumulative DI to eliminate the original Park-Ang non-convergence problem at upper and lower limits. However, un-coherently with the dynamic behaviour of reinforced concrete members, the modified model can only deal with the Park-Ang DI non-convergence problem when seismic damage is caused by monotonic loading effect.

In this paper, a modification is proposed for the original Park-Ang damage index to eliminate its non-convergence problem at the upper and lower bounds. Then, the probabilistic seismic demands of 8 representative regular highway bridge samples are calculated with the increment dynamic analysis (IDA) method [12] in the OpenSees environment. Finally, a general and practical probabilistic seismic demand model for regular bridges is developed, which combines: a) a ground-motion intensity measure (IM), which is the spectral acceleration at the fundamental period with 5% damping, and b) one engineering demand parameter (EDP), which is the modified Park-Ang damage index..

2. Modified Park-Ang damage index

The Engineering Demand Parameters (EDPs) are structural response quantities that can be used to predict the damage to structural and non-structural components and systems. The damage indices may be based on the results of a nonlinear dynamic analysis, the measured response of a structure during an earthquake, etc. In most cases, the damage indices are dimensionless parameters that should range from 0 for the undamaged (elastic) state to 1 for the collapsed state of a structure, and the intermediate values provide some measure of the degree of damage.

Consistently with the dynamic behaviour, Park and Ang [10] expressed the seismic structural damage as a linear combination of the damage caused by excessive deformation and that contributed by repeated cyclic loading. In terms of damage index, this quantity is represented as:

$$D_{PA} = \delta_M / \delta_u + \beta \int dE / (Q_y \delta_u) \quad (2)$$

Where, δ_M is the maximum deformation under earthquake, δ_u is the ultimate deformation under monotonic loading, Q_y is the calculated yield strength, dE is the incremental absorbed hysteretic energy, and β is the coefficient of the cyclic loading effect (a function of the structural parameters).

As previously noted, the original Park-Ang damage index (DPA) does not converge at its upper and lower limits, that is: (a) damage index is greater than 0 when structures are loaded within elastic range, thus sustain no damage; (a) damage index is greater than 1 while structures are loaded monotonically to failure [11,13].

A large set (272) of cyclic test data [14-16] of flexure dominant reinforced concrete bridge piers were considered in this study to verify the actual estimation of the damage of DPA at failure. These experimental data were carefully selected from a large set of test specimens; only those with a clearly

observed abrupt failure or an identifiable gradual failure on the envelope curve were included. The failure of the bridge piers were defined as the first occurrence of one of the following events: the buckling of a longitudinal bar, the fracture of a transverse reinforcement, the fracture of a longitudinal bar, or the loss of the axial-load capacity. The yielding of the reinforced concrete bridge piers were defined as the first yielding of the tension reinforcement. The ultimate deformation $\delta_{u,c}$ under cyclic loading was considered 0.62 times the ultimate deformation δ_u under monotonic loading [17]. The parameter β was calculated according to the regression expression obtained using experimental results [15]. The absorbed hysteretic energy was the area of all closed cycles up to the cycle of the failure point. The capacity values of DPA for the available test data at yielding and at failure is shown in figure 1 and figure 2. It can be observed that the capacity values of DPA do not provide the value 0 in the case of yielding and 1 in the case of failure and it confirms that the original Park-Ang damage index DPA does not converge at the lower and upper bound.

In order to eliminate that problem, Jiang, Chen and Chen (2011) [11] proposed a modified form for the Park-Ang, as below:

$$D_{PA} = (1 - \beta) \frac{\delta_M - \delta_c}{\delta_u - \delta_c} + \beta \frac{\int dE}{Q_y (\delta_u - \delta_y)} \quad (3)$$

Where, δ_c is deformation at initial cracking of outer concrete and β is different from combination coefficient proposed by Park-Ang.

The Jiang et al. damage index seems to solve the Park-Ang converge problems only for seismic damage caused by monotonic loading effect. Hence, a modification for the original Park-Ang damage index is proposed in this paper.

The modified Park-Ang damage index DMPA, which defines the structural damage as a combination of the damage caused by the normalised plastic deformation ζ and the normalised hysteretic energy dissipated η , is given as follows:

$$D_{MPA} = 1 - (1 - \zeta) / (1 + \eta) \quad (4)$$

Where, the two normalised parameters ζ and η are calculated as follows:

$$\zeta = (\delta_M - \delta_y) / (\delta_{u,c} - \delta_y) \quad (5)$$

$$\eta = \int dE / (Q_y \delta_{u,c}) \quad (6)$$

Where, $\delta_{u,c}$ is the ultimate deformation under cyclic loading.

As can be seen from figure 1 and figure 2, the new modified damage function is 0 at the beginning of yielding and it is 1 at failure and increases as the damage increases. Table 1 lists the capacity values of the two DIs in the plastic deformation range. It can be seen that the coefficient of variation of modified DI is smaller than the original one.

Table 1. Statistical values of the damage indexes at different limit states.

Plastic state	Number of specimens	D_{MPA}			D_{PA}		
		Mean	STD	COV	Mean	STD	COV
Cover spalling	114	0.45	0.21	0.47	0.30	0.17	0.55
Maximum strength	272	0.67	0.18	0.22	0.68	0.37	0.54

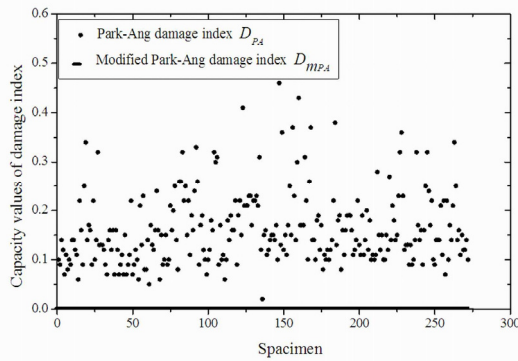


Figure 1. 272 cyclic test specimens at yielding.

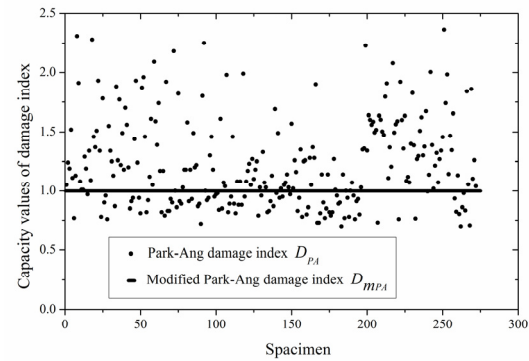


Figure 2. 272 cyclic test specimens at failure.

3. Regular bridges numerical modeling

This paper considers only regular bridges, which are bridges whose seismic response is controlled by the fundamental vibration modes. The specific definition for regular highway bridges are detailed in current Chinese seismic code [18]. Based on the characteristics of regular bridges, the seismic demand analysis of regular bridges can be performed using a simplified single-pier model [19]. The simplified single-pier model in this study is schematically represented in figure 3. The other seven bridges were derived from it by enlarging and shortening the piers to vary the lateral stiffness of the bridges or by increasing and reducing the superstructure mass, which varies the vibration periods of the fundamental response modes of the bridges. A total of 8 bridge samples are analysed in this paper.

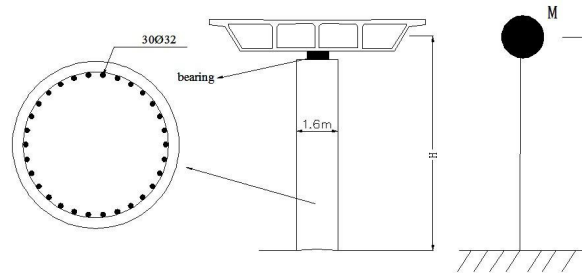


Figure 3. The simplified single-pier model of representative regular bridges.

Finite element models of the analysed bridges are developed using the OpenSees software [20]. According to the modelling procedure [21], the superstructure of the bridge is modelled with a lumped mass, and the pier is modelled using the fibre-section beam-column element. The longitudinal column reinforcement is modelled using the reinforcing steel material, which accounts for isotropic hardening, diminishing yield plateau and the Bauschinger effect. The OpenSees material concrete04 is used to model the confined concrete and the unconfined concrete. The yield strength/deformation and the ultimate deformation under cyclic loading are computed with a reversed cyclic pushover. The basic parameters of the eight representative regular bridges are listed in Table 2.

Table 2. Basic parameters of the representative regular bridges.

No.	T_1 (s)	H (m)	M (kg)	Q_y (kN)	δ_y (mm)	$\delta_{u,c}$ (mm)
1	0.48	7.5	500000	933	37.5	300
2	0.75	10.0	500000	751	68	500
3	0.95	10.0	800000	858	68	500
4	1.10	10.0	1000000	937	68	500
5	1.30	12.5	800000	676	97.5	650
6	1.50	12.5	1000000	729	97.5	650
7	1.70	15.0	800000	572	132	800
8	2.0	15.0	1000000	621	132	800

4. Incremental dynamic analysis

In this paper, the ground-motion records that are used to perform the IDA are selected from the PEER Strong Motion Database and Country Strong Motion Network Centre. During the seismic demand estimation, the record-to-record variability of seismic excitations must be considered. Therefore, 2390 horizontal earthquake records from 57 seismic events (such as Northridge, Chi-Chi, Wenchuan, etc.) are considered. All records are characteristic of non-near-fault motions ($R > 15$ km). The site categories of the selected records are classified based on the average shear wave velocity to a depth of 30 m according to the Chinese seismic code: type I ($V_{s30} > 510$), type II ($260 < V_{s30} \leq 510$), type III ($150 < V_{s30} \leq 260$) and type IV ($V_{s30} \leq 150$) [22]. The number of different site categories of records in this paper are as follows: type I = 514, type II = 1214 and type III = 662. Few records belong to type IV, and they are not in the scope of this paper.

Within the bridge Engineering community, PGA and S_a are the most commonly adopted IMs [23]. Although the PGA is a much more familiar IM, it is an optimal IM only for stiff structures with very short vibration periods. This paper considers only regular bridges, which are bridges whose seismic response is controlled by the fundamental vibration modes. The fundamental period of regular bridges studied in this paper ranges from 0.48 s to 2.0 s. Hence, the widely accepted spectral acceleration at the fundamental period would then be optimal. In this paper, the spectral acceleration at the fundamental period with 5% damping, S_a , was selected as the IM. The engineering demand parameter in this study is the modified Park-Ang damage index, which is defined in Equation 4. The proposed EDP, which considers both the influence of plastic deformation and the dissipated hysteretic energy, appears to provide realistic measures of the structural damage.

The nonlinear time-history response analyses of the 8 representative regular highway bridge samples for each of the selected 2390 earthquake records at three different site conditions were performed. The 2390 records were scaled with the S_a range of 0.2 g to 1 g with a step increment of 0.05 g, so each record was considered 17 times for each bridge sample. Therefore, 325040 nonlinear time-history analyses were conducted in this study.

5. Probabilistic seismic demand model

The probabilistic seismic demand model is a mathematical expression that relates the ground-motion IM (S_a) to the structure specific demand measure (DI). This model is used to predict the mean value of the demand, which is μ , at a specific IM value.

The unknown standard deviation and the model parameters require a database, which consists of the IDA results of designated structures that are subjected to the selected ground-motion records. Although the IDA is a proper method to produce the intended database and it can describe the behaviour of the structure in different performance levels, it is time-consuming and high-cost and requires many operations. Thus, a simplified and reliable empirical expression of the EDP is proposed as follows.

Most probability distributions are for random variables whose range of values is unlimited in one or both directions. The beta distribution, one of the few distributions with finite lower- and upper-bound values, is used in this paper [24]. In the standard beta distribution, the variate is bounded between 0 and 1.0. Its probability density function is given by:

$$f(x) = \frac{1}{B(q, r)} x^{q-1} (1-x)^{r-1} \quad 0 \leq x \leq 1.0$$

$$= 0 \quad \text{otherwise} \quad (7)$$

Where, q and r are the parameters of the distribution, and $B(q, r)$ is the beta function as follows:

$$B(q, r) = \int_0^1 x^{q-1} (1-x)^{r-1} dx \quad (8)$$

The data from the IDA curves were analysed to obtain the conditional probability distribution model of the EDP at the given IM level. The authors only present the frequency diagram of the EDP of

the No. 4 bridge, which was subjected to the records with $Sa = 0.3$ g (see figure 4). The standard beta distribution is assumed for the conditional probability distribution of the EDP. The assumed distribution model was verified using a goodness-of-fit test with a P-P plot. It can be seen from the P-P plot in figure 5 that the maximum discrepancy between the assumed beta model and the observed data is less than 0.08. Hence, the EDP has a reasonable standard beta distribution when it is conditioned on the IM.

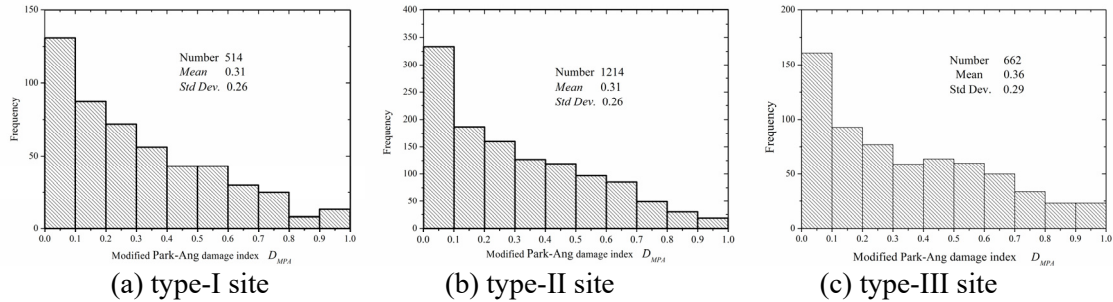


Figure 4. Frequency diagram of the EDP of No. 4 bridge subjected to the records with $Sa = 0.3$ g.

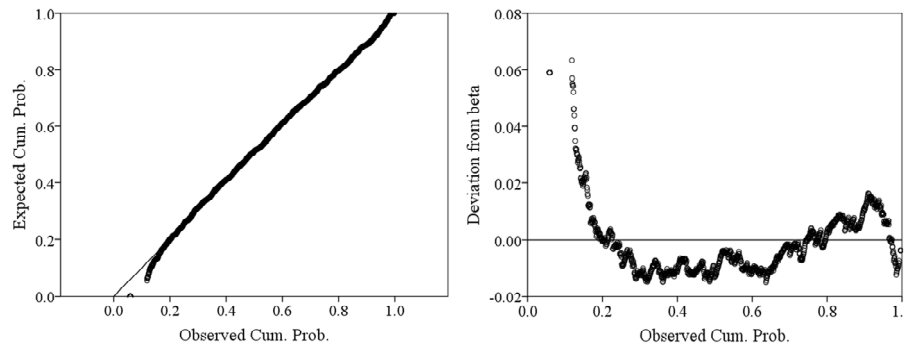


Figure 5. Standard beta P-P plot of the EDP of No. 4 bridge subjected to the records with $Sa = 0.3$ g.

The mean EDP value μ_D , which was derived from the conducted IDA for 8 regular bridges. It can be observed that the mean EDP value is slightly larger at the type-III category because the expected spectral acceleration at type-III sites for an equivalent earthquake is more likely amplified in the period range of the considered bridge modes. The simplified expression of the mean EDP value based on the type-III site provides conservative values. Based on the statistical results, the simplified expression to estimate the mean EDP value is proposed as follows:

$$\mu_{D_{MPA}} = a + b(Sa) + c(Sa)^2 \quad (9)$$

Where, Sa is the spectral acceleration at the fundamental period, and a , b , and c are site-independent constants. The parameters a , b , and c are computed using a nonlinear regression analysis with the least square method for each regular bridge. The resulting values of these parameters depend on the fundamental period T . It is important to note that the fundamental period ranges from 0.48 s to 2.0 s. The proposed simplified empirical expression for the mean EDP value is:

$$\mu_{D_{MPA}} = (0.14 - 0.63T + 0.28T^2) + (-1.14 + 4.54T - 1.45T^2)(Sa) + (1.33 - 3.14T + 0.93T^2)(Sa)^2 \quad (10)$$

To check the accuracy of Equation 10, the scatter diagram that was used to compare the EDP value using the empirical expression and using the IDA method is shown in figure 6. Each point on the plot represents the mean EDP value for a sample bridge and a given IM level. The diagonal line across the plot implies equal μ_D estimates from the empirical expression and the IDA method. It can be observed that these data are uniformly distributed on both sides of the diagonal. The statistical values of the ratio of Equation 10 and IDA obtain values are shown in table 3. All these data demonstrate that the Equation 10 provides good estimation of the mean EDP. In addition, the simplified empirical expression method uses less calculation than the IDA method.

Table 3. Statistical values of the ratio of Eq.10 obtain values and IDA obtain values.

Site	Number of data	Mean	STD	COV
TypeI	136	1.14	0.17	0.19
TypeII	136	1.10	0.15	0.16
Type III	136	1.01	0.15	0.15

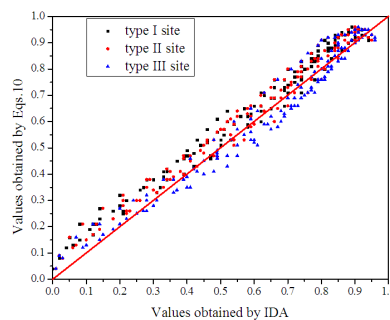


Figure 6. Scatter plot that compares the mean EDP value that is estimated using the empirical expression and that using the IDA method.

6. Conclusions

A modified Park-Ang damage model based on the combination of plastic deformation and absorbed hysteretic energy is proposed in this paper. The probabilistic seismic demand model of regular bridges has been investigated. From this study, it can be concluded that: 1) The modified model eliminates the non-convergence problem of the original Park-Ang damage index. 2) The probability distribution model of the modified Park-Ang damage index demand of regular bridges obeys a standard beta distribution when it is conditioned on the level of S_a . 3) A simplified and reliable general PSDM for the regular bridges is a significant step toward developing a probabilistic seismic performance design and assessment procedure. 4) Subsequent work on the probabilistic seismic demand analysis of regular highway bridges should focus on expanding the bridge portfolio to fully include a variety of structural physical parameters (such as the section reinforcement and the support form).

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