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Publication date:
1995

Document Version
Publisher's PDF, also known as Version of record

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STRUCTURAL RELIABILITY THEORY
PAPER NO. 138

To be presented at "Soil Dynamics and Earthquale Engineering", Crete, Greece, May 24-26, 1995

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Prediction of global damage and reliability based upon sequential identification and updating of RC structures subject to earthquakes

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Abstract

The paper deals with the prediction of global damage and future structural reliability with special emphasis on sensitivity, bias and uncertainty of these predictions dependent on the statistically equivalent realizations of the future earthquake. The predictions are based on a modified Clough-Johnston single-degree-of-freedom (SDOF) oscillator with three free parameters which are calibrated to fit the displacement response and the damage development in the past earthquake. As a damage indicator, the maximum softening index defined on the variations of the first period of a structure during an earthquake by Çakmak et al. is used. For reliability considerations the failure event is defined as the first passage of a certain critical level. In the lack of sample tests, results for comparisons are generated artificially having a more accurate but time consuming finite element model run in the SARCOF program developed by Mørk (1992). The performance of the model is illustrated by a 4-storey, 1-bay RC frame. An initial earthquake implying a certain maximum softening of the structure is applied. Next, the SARCOF model and the hysteretic SDOF oscillator with calibrated parameters are exposed to 1000 statistically equivalent realizations of a stochastic earthquake model, and the mean value and variance functions of the damage indicator conditioned on a certain damage in the first earthquake are calculated statistically. The estimates obtained using the finite element model and the SDOF oscillator are observed to be very close. Hence, the reliability estimates of the two models are approximately the same.

Key Words: Earthquake excitation, RC-structures, damage prediction, structural reliability, hysteretic models for RC.

1 Introduction

Global damage indicators are response quantities characterizing the damage state of the structure after an earthquake excitation, and such can be used in decision-making of various draft proposals during the design phase, or in post-earthquake reliability and repair problems of damaged structures. In serving these purposes, the global damage indicator should be observable for practical purposes, be a non-decreasing function of time unless the structure is repaired or strengthened, provide a unique failure criterion to separate the safe states from the unsafe ones, and, possess Markov property so that post-earthquake reliability estimates for a partly damaged structure can be estimated solely from the latest recorded value of the damage indicator. Nielsen and Çakmak [6].

The maximum softening damage indicator, introduced by DiPasquale and Çakmak [1] as a global damage indicator, measures the maximum relative reduction of the eigenfrequency of an equivalent linear SDOF oscillator with slowly varying stiffness properties, displaying the combined damaging effects of the maximum displacement ductility of the structure during extreme plastic deformations and the stiffness deterioration in the elastic regime, the latter effect being referred to as final softening. Köylüoğlu et al. [3] proposed a modified Clough-Johnston hysteretic oscillator as a simple model describing the behaviour of the first eigenmode. A novelty was the modelling of the elastic fraction of the restoring force as a decreasing function of the accumulated numerical plastic displacements on the hysteretic component displaying the transition from elastic to plastic behaviour as cracks and damage occur. The circular eigenfrequency, damping ratio and modal participation factor of the first mode of the undamaged structure were assumed to be known, measured before the arrival of the first earthquake from non-destructive vibration tests or by means of structural analysis. The other two parameters of the hysteretic model were identified and updated after each earthquake. Upon suitable calibration of the two hysteretic parameters, the model was observed to be capable of predicting the displacement response and the development of the maximum softening compared to the recorded response of shake table experiments on frame RC-structures, Çecen [2].

In the present paper, the indicated SDOF-hysteretic model has been further developed and the strength is assumed to deteriorate at the expense of the introduction of one further free parameter. The aim of the paper is to estimate the uncertainty of damage predictions of the model by means of Monte Carlo simulation with respect to the realizations of statistically equivalent earthquakes. As a reference, the original frame is modelled using finite elements and the structural analysis is performed by the SARCOF-program developed by Mørk [5], which has previously been proven to be able to reproduce the results of Çecen [2] with good accuracy, when cracked moment of inertias are employed to describe the bending stiffness of the concrete beams. The program provides the development of instantaneous softening via an eigenvalue solver subroutine applied to the instantaneous global stiffness matrix. The objection that the SARCOF program, being another model, may not represent reality is taken as being irrelevant to the subject of study. Thus, the SARCOF program is used to produce some time series of the displacement and damage development, which may have been measured, and the simplified model is only calibrated during the past earthquake to reproduce the response time series in the future earthquake.

2 Hysteretic model for SDOF oscillator

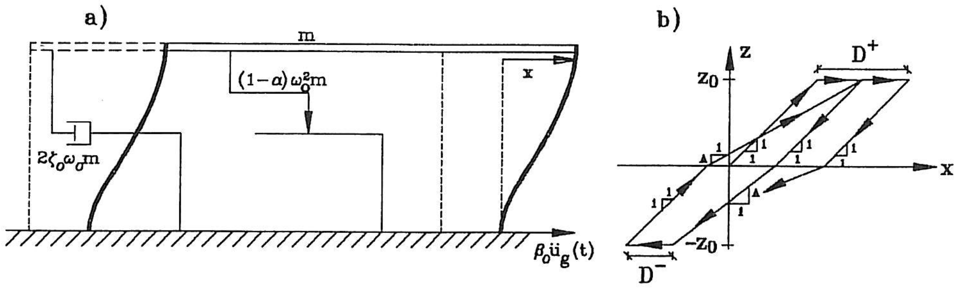

Figure 1: *a) SDOF hysteretic oscillator model b) Clough-Johnston hysteretic model.*

The equations of motion of the first mode is modelled by the following coupled differential equations, Köylüoğlu et al. [3]

$$\ddot{x}(t) + 2\zeta_0\omega_0\dot{x}(t) + \omega_0^2\Big[\alpha(t)x(t) + (1-\alpha(t))z(t)\Big] =$$
$$-\beta_0\ddot{u}_g(t) \quad , \quad t > t_0 \quad , \quad x(t_0) = \dot{x}(t_0) = 0 \tag{1}$$

$$\dot{z}(t) = \Big[H(z)\Big(A(t)H(\dot{x})(1 - H(z - z_0(t))) + H(-\dot{x})\Big) +$$
$$H(-z)\Big(A(t)H(-\dot{x})(1 - H(-z - z_0(t))) + H(\dot{x})\Big)\Big]\,\dot{x}(t) \quad , \quad z(t_0) = 0 \tag{2}$$

$$\dot{D}(t) = [H(\dot{x})H(z - z_0(t)) - H(-\dot{x})H(-z - z_0(t))]\,\dot{x}(t) \quad , \quad D(t_0) = D_0 \tag{3}$$

$$\alpha(t) = \alpha(D(t)) = \frac{1}{(1 + \frac{D(t)}{2z_0(t)})^{n_0}}, \qquad z_0(t) = z_0(D(t)) = \frac{z_{0,0}}{1 + n_1\frac{D(t)}{z_{0,0}}} \tag{4}$$

$$A(t) = \frac{z_0(t)}{z_0(t) + D(t)}, \qquad H(x) = \begin{cases} 1 & , \quad x \geq 0 \\ 0 & , \quad x < 0 \end{cases} \tag{5}$$

The first modal coordinate $x(t)$ can be defined as the top storey displacement of the structure relative to the ground surface if the mode shape is suitably normalized. The linear circular eigenfrequency, ω_0, the damping ratio, ζ_0, and the mode participation factor, β_0, of the first mode are assumed to be known before the arrival of the first earthquake. $\ddot{u}_g(t)$ indicates the horizontal earth surface acceleration signal and the earthquake starts at the time $t = t_0$. $\alpha(t)$ is the elastic fraction of the restoring force, which is assumed to decrease as a function of the accumulated plastic deformation $D(t)$. $z(t) \in [-z_0(t), z_0(t)]$ is the hysteretic component, which is modelled using the Clough-Johnston hysteretic model, and $z_0(t)$ signify the instantaneous strength (yield level) of the oscillator, which is deteriorating from its initial value $z_{0,0}$ as the accumulated plastic deformations evolve. This deterioration was not included in the first formulation of the model by Köylüoğlu et al. [3] and seems to provide significant improvements at the expense of the extra parameter $z_{0,0}$. The stiffness degrading hysteretic constitutive law of the model can be represented as shown in Figure 1.b. The Clough-Johnston model deals with the stiffness degradation by changing the slope $A(t)$ of the elastic branches as the accumulated plastic deformations, $D^+(t)$ and $D^-(t)$ at positive and negative yielding, increase as shown in Figure 1.b. $D(t) = D^+(t) + D^-(t)$ are the total accumulated plastic deformations. For loading branches, the slope $A(t)$ is selected such that the elastic branch always aims at the previous unloading point with the other sign. At unloadings, the slope is 1. D_0 is the initial value of the total accumulated damage which is zero before the first earthquake hits and is assumed to be determined from previous earthquake and displacement response records for the succeeding earthquakes. $H(x)$ is the Heaviside stepfunction.

The hysteretic parameters $z_{0,0}, n_0$ and n_1 are to be identified from the experienced excitation and the displacement response time series with a suitable optimization method. The Clough-Johnston hysteretic model was originally designed for reinforced concrete beams. The differential description of the model, applied herein, is due to Minai and Suzuki [4].

3 Damage measures and prediction of damage and reliability

The instantaneous softening, $\delta(t)$, of a structure is defined as, Çakmak et al. [1].

$$\delta(t) = 1 - \frac{T_0}{T(t)} \qquad (6)$$

where T_0 is the first period of the equivalent linear structure and $T(t)$ is the first period of the equivalent linear structure with slowly varying stiffness characteristics during an earthquake excitation, which is estimated from the excitation and displacement response time series of the experienced earthquake. The maximum softening damage indicator, δ_M, is the maximum of $\delta(t)$ during the seismic excitation.

Consider the SDOF hysteretic model where $D(t)$ is related to an average equivalent slope of the hysteretic loops. This is choosen as the slope of the line through the extreme points, see Figure 1.b

$$\overline{m}(t) = \frac{2z_0(t)}{2z_0(t) + D(t)} \tag{7}$$

The circular eigenfrequency of the equivalent linear oscillator then becomes $\omega(t) = \omega_0 \sqrt{\alpha(t) + (1 - \alpha(t))\overline{m}(t)}$, resulting in the estimated instantaneous softening

$$\hat{\delta}(t) = 1 - \sqrt{\frac{2z_0(t)}{2z_0(t) + D(t)}\Big(1 - \alpha(t)\Big) + \alpha(t)} \tag{8}$$

As seen from (8), $\hat{\delta}(t)$ is non-decreasing during a seismic event and fully correlated to $D(t)$. The proposed hysteretic model for the SDOF system is defined by six parameters, namely, ζ_0 , ω_0 , β_0 , $z_{0,0}$, n_0 and n_1. ζ_0 , ω_0 , β_0 are measured on the undamaged structure (here obtained from the SARCOF program), whereas $z_{0,0}$, n_0 and n_1 are estimated from the following least square criterion

$$\min_{\hat{z}_{0,0},\hat{n}_0,\hat{n}_1} \; w_1 \sum_l (\hat{\delta}_l - \delta_l)^2 + w_2 \sum_k (\hat{x}_k - x_k)^2 \tag{9}$$

where $\hat{\delta}_l = \hat{\delta}(l\Delta t)$ is the instantaneous softening at the l th timestep obtained by the hysteretic model with the parameter $\hat{z}_{0,0}$, $\hat{n}_0$ and $\hat{n}_1$, and δ_l is the corresponding measured quantity (here from SARCOF realizations). Similarly, $\hat{x}_k$ and x_k are the estimated and measured displacement, respectively. w_1 and w_2 are positive weights, which are assigned such that displacement and instantaneous softening contributions in the error are approximately equal. Furthermore, large oscillations are weighted higher than small oscillations by excluding parts of the time series.

4 Uncertainty of the reliability estimates

In an earlier study it was shown that the maximum softening values of structures subject to consecutive earthquakes form a Markov chain, Köylüoğlu et al. [3]. This means that both the damage process and structural reliability measured using the maximum softening value in future earthquakes can be predicted using the a SDOF hysteretic model if only the damage process $\delta(t)$ is updated after each seismic event, i.e. the terminal value in the previous earthquake via updated model parameters $z_{0,0}$, n_0 and n_1. The reliability is normally estimated from the transitional probability density function. Assume that the structure has been damaged to a level providing the maximum softening δ_1 in the first earthquake. The statistical distribution of the damage in the next future earthquake can then be estimated by Monte Carlo Simulation, where the structure is exposed to a sufficiently large number, n, of realizations of the design ground surface acceleration process, $\ddot{u}_g(t)$. The damage development $\delta_2(t)$ from the initial value δ_1 and the top storey displacement $\hat{x}(t)$ is next calculated by both models for these realizations. In principle the probability of failure, $P_f(\delta_1)$, that the damage exceeds a certain critical level δ_0 is requested. The reliability is normally estimated from the transitional probability density function. However, in the practice, only the conditional mean value $E[\delta_2(t)|\delta_1]$ and the conditional variance $Var[\delta_2(t)|\delta_1] = E[\delta_2^2(t)|\delta_1] - (E[\delta_2(t)|\delta_1])^2$ functions can be estimated accurately. These are estimated using both the finite element model with the SARCOF program and the simple hysteretic oscillator model. The quality of the damage predictions of the latter model in statistical measures is then evaluated through comparisons.

5 Numerical examples

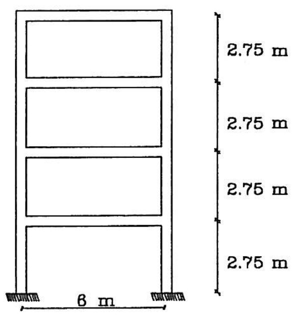

Figure 2: *4-storey, 1-bay reinforced concrete frame.*

The 4-storey, 1-bay reinforced concrete structure shown in Figure 2 is used to illustrate the performance of the method. The ground surface acceleration process is modelled as a non-stationary white noise (shot noise) filtered through a Kanai-Tajimi filter. The damping ratio of the filter is 0.3, and the circular eigenfrequency is set to 8.8 $\sec^{-1}$. The modulation function attains its maximum after 3 sec and stays constant for 15 sec and then decays exponentially. The intensity of the peak acceleration time series is chosen so that the peak acceleration is $0.5g$ in all the realizations in the 2nd earthquake. The equations of motion are solved using a 4th order Runge-Kutta scheme for both the SARCOF model and the simple hysteretic oscillator. The timestep is selected as $\Delta t = 0.008$ sec, where it has been checked that no drift occurs in the simulated signal due to numerical instability. The parameters have been selected so that for the uncracked structure one has $\omega_1 = \omega_0 = 8.25 \sec^{-1}$, $\omega_2 = 26.25 \sec^{-1}$ $\zeta_2 = \zeta_1 = \zeta_0 = 0.05$ and $\beta_1 = \beta_0 = 1.33$. The unconstrained minimization problem (9) is solved by trial-and error. More efficient optimization methods based on gradient calculations fails since the gradients are non-continuous in the present problem as a consequence of the non-analytic right-hand sides of the differential equations (2) and (3). Furthermore, there are several local minimums in the optimization problem. The parameters for the hysteretic model are calculated as shown in table 1.

δ_1	0.069	0.114	0.182	0.217	0.258	0.307	0.351	0.405
$z_{0,0}$ [mm]	48.0	48.0	49.0	55.0	80.0	80.0	94.0	103.0
n_0	1.80	1.50	1.20	1.50	1.40	1.45	1.50	1.40
n_1	0.40	0.30	0.40	0.50	0.52	0.60	0.45	0.48

Table 1: *The estimates parameters defining the hysteretic model.*

The performance of the calibrated hysteretic oscillator in the first earthquake is shown in Figure 3 for a sample with δ_1=0.307. It is seen that the model fits very well to the reference data for both the damage level and the top storey displacements, Figure 3a and 3b. The damage predicted in a second earthquake by the hysteretic oscillator after calibration of the hysteretic parameters at the end of the 1st earthquake is shown (continuous line) along with the reference data (dashed line) in Figure 3c. As seen, the model gives a good estimate of the damage.

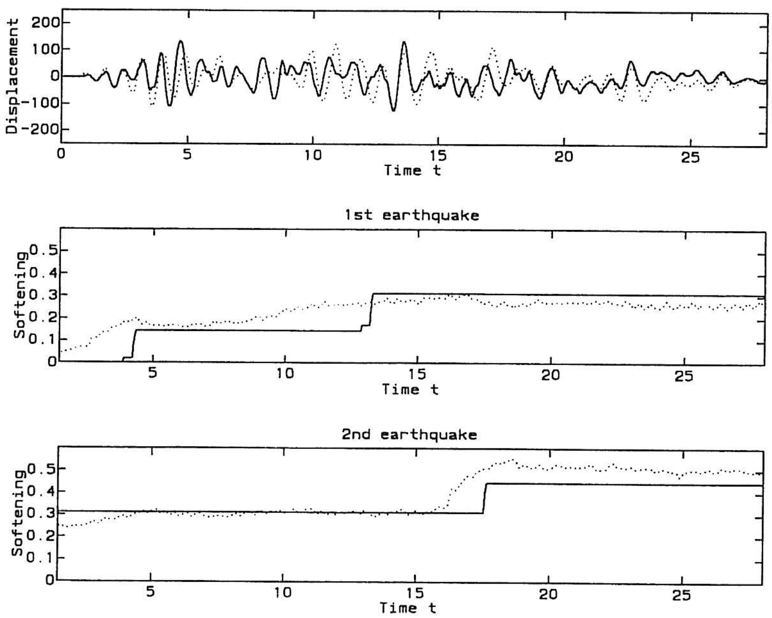

Figure 3: *The performance of the calibrated hysteretic model for an excitation realization in the 1st and 2nd earthquake. a) Predicted top storey response. b) Predicted damage in earthquake 1. c) Predicted damage in the 2nd earthquake.*

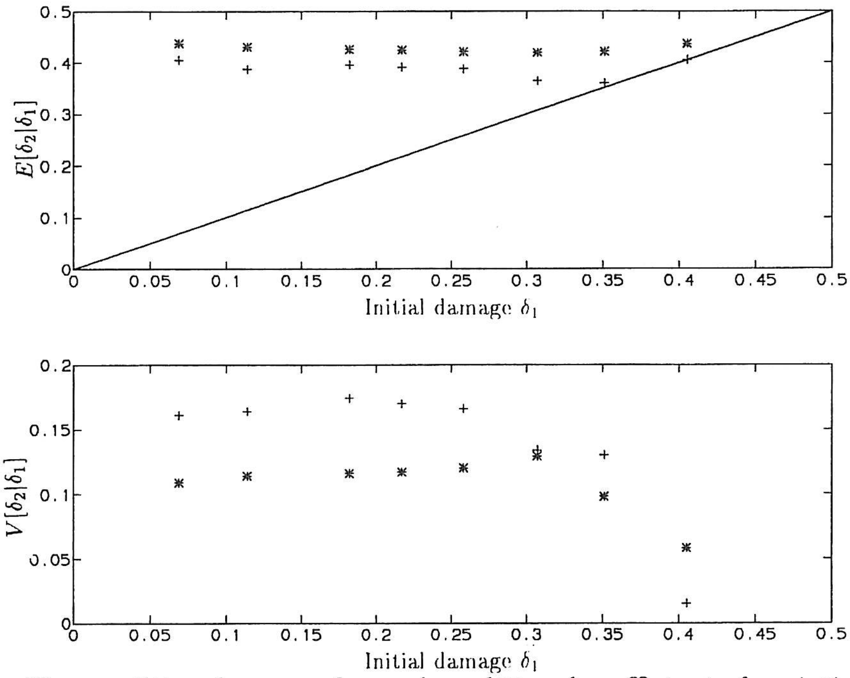

Figure 4: *The conditional mean value and conditional coefficient of variation of the maximum softening damage indicator. $*$: Reference data obtained via SARCOF and $+$: Predictions by the hysteretic model.*

The statistical analysis is carried out by simulating $n = 1000$ independent realizations of the 2nd earthquake by the SARCOF model and the calibrated hysteretic oscillator. The estimated conditional mean value and the conditional standard deviation of the predictions are shown in Figure 3. The estimates of the conditional mean value are observed to be slightly underestimated and the conditional variances are somehow overestimated, if the initial damage level δ_1 is small. This will result in conservative reliability estimates. At higher values of δ_1 the correspondence is sufficiently better, and practically unbiased. Since the parameters $z_{0,0}$, n_0 and n_1 in the case of small values of δ_1 are calibrated primary from the elastic response of the structure (small damage). Then, it is not likely that the oscillator is able to predict any heavily damaged plastic states in the next strong earthquake.

6 Conclusions

A simplified SDOF hysteretic oscillator model for prediction of structural damage of RC-structures exposed to earthquake excitations is investigated. The model has three parameters that are updated sequentially at the end of each major earthquake. The study focuses on the uncertainty of reliability estimates provided by the simplified model with respect to the realizations of statistically equivalent future earthquakes. The lower order moments of such conditional uncertainty are tested by means of a Monte Carlo simulation approach at different damage levels. The reference sample set for comparisons was generated by the SARCOF program. In the studied examples it is observed that the hysteretic SDOF oscillator yields about 10 per cent higher values for the failure probability with better results as the damage level in the first earthquake is increased. This reflects more accurate estimation of the three parameters of a hysteretic model at large damage levels. It should be noted that the cpu time spent for the estimation of the conditional means and variances using the SDOF model is 100 times less than that of the SARCOF program.

7 Acknowledgement

The present research was partially supported by The Danish Technical Research Council within the project: Response and Reliability of Partially Damaged Reinforced Concrete Structures subject to Random Dynamic Loads.

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