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Assessment of Damage in Seismically Excited RC-Structures from a Single Measured Response

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Abstract *A method has been developed for the localization of structural damage of substructures of seismically excited RC-structures using only the ground surface acceleration time series and a single response time series. From the response, the smoothed two lowest eigenfrequencies are estimated. The distribution of local damage is then performed in such a way that these smoothed eigenfrequencies are reproduced. The local damage indicators for a certain substructure are defined as the average reduction of the stiffness matrix of the initial undamaged substructure. These damage indicators are identified by a sequence of substructures, where two new substructures are introduced at each level, so that the smoothed eigenfrequencies are reproduced at each level. The method is applied to simulated data of a 1-bay, 2-storey RC-frame and a 1-bay, 4-storey RC-frame generated by a finite element programme developed for RC-frames, which also admits an estimation of local damage. Based on the response time series calculated by the finite element programme, the corresponding local damages are next calculated by the present method. The method is investigated at different intensities of the earthquake and upon comparison with the finite element predictions it is found that the method is very efficient in localizing the damage when the damage level is sufficiently high. Furthermore, it has been experienced that the method seems to get slightly more inaccurate with increasingly structural complexity, i.e. the damage localization can only be driven to a certain limit due to the limited amount of information in the single response time series.*

Keywords: Damage, Localization, System Identification, Earthquakes, Finite Element Model.

Nomenclature

ω	Circular eigenfrequency.
T	Eigenperiod.
δ	Damage indicator.
$\mathbf{K}$	Stiffness matrix.
$\mathbf{M}$	Mass matrix.
$\mathbf{\Phi}$	Mode shape matrix.
t	Time.
E	Modulus of elasticity.
I	Moment of inertia.
A	Cross-section area.

1 Introduction

Experiences from past earthquakes in the last decade have shown a growing need for methods for localization and quantification of damage sustained by RC-structures during earthquakes. Visuel inspection and field testing can be used to locate and measure the damage state of an RC structure. However, a much more attractive method is measuring of the structural response at a given location of the structure. From this response time series, changes of dynamic characteristics such as changes in stiffness and damping can be identified, and therefore, they are suitable for calculation of local decrease in stiffness which is used in the calculation of damage. Damage in a substructure is here defined as the average relative reduction of the stiffness matrix of the considered substructures that reproduces the two lowest eigenvalues of the global structure. The problem to be solved is then to identify the stiffness matrix that reproduces the measured eigenvalues. However, normally the number of elements where damage can occur is much larger than the number of estimated eigenfrequencies causing an underdetermined system giving multiple solutions to the local damage.

During the last decade, assessment of damage in structures has attracted much interest from various researchers. Until now, the focus has mainly been put on assessment of damage in steel structures where the damage is in the form of fatigue cracks, poor joints, etc. Common for all such approaches is that it is sought to establish a relation between measured modal quantities such as circular eigenfrequencies and eigenvectors and local damage indicators which are often defined from stiffness reduction of members or partitioned substructures. All the methods are more or less based on an assumption that a linear expansion (1st order pertubation analysis) of modal properties from the undamaged state is valid. This is a plausible assumption as long as the changes in the modal quantities are small ($< 20\%$). However, this is not the case for RC-structures during e.g. an earthquake, where the changes in the modal parameters are considerable (often $> 50\%$). These facts require further development of the proposed methods to take into account the heavy non-linear behaviour before they are appliable to severely damaged RC-structures.

A method especially developed for large changes in eigenfrequencies was presented by Skjærbæk et al. [10]. The method is based on a finite element model of the structure which is sequentially partitioned into two substructures for which an average damage is calculated. It has proven to work very well for a 1-bay, 4-storey RC-frame damaged only in a single beam. Furthermore, perfect data were used and the additional problems by using imperfect and uncertain data were not treated.

The purpose of this paper is to illustrate how this method works for seismically excited RC-structures, where the structure will be continously damaged, and illustrate the problems with imperfect data.

2 Measurement Procedure

During a severe excitation such as a strong motion earthquake, the structure will be sequentially damaged due to cracking, debonding, crushing of concrete and post-yielding of reinforcement bars. The circular eigenfrequencies, $\omega_i(t)$, of the time-varying structure will vary rapidly as the structure enters and leaves the plastic regime. To extract the long-term tendency of this quantity, which displays the time-variation of the structural parameters due to damages, a smoothing, denoted $\langle\omega_i(t)\rangle$, becomes necessary. This is equivalent to modelling the long-term development of the actual structure by an equivalent linear time-varying replacement with the circular frequencies $\langle\omega_i(t)\rangle$. Based on a single input single output measurement, it will normally not be possible to estimate more than $\langle\omega_1(t)\rangle$, $\langle\omega_2(t)\rangle$ for seismicly excited structures. $\langle\omega_i(t)\rangle$ can be measured using time-windowing ARMA-models, DiPasquale and Çakmak [2], time-averaging FFT, Mullen et al. [5] or discrete wavelet transforms, Micaletti et al. [4]. These methods give somewhat different results, since they highly depend on the windowing length, order of ARMA-model (i.e. the number of degrees of freedom of the applied equivalent linear system), etc. Consequently, the estimates of $\langle\omega_i(t)\rangle$ should be considered as uncertain quantities, where the uncertainty increases with the mode number. Figure 2 shows computer simulated realizations of the 4 storey 1-bay structure shown in figure 1 of the horizontal base acceleration $\ddot{u}_g(t)$, the horizontal top storey displacement $x(t)$ response relative to the ground surface, and the development in the two lowest eigenfrequencies are shown using the program SARCOF, Mørk et al. [6]. $\omega_i(t)$ are shown as a full line, and $\langle\omega_i(t)\rangle$ as a dashed line. In the programme, the smoothed value $\langle\omega_n(t_1)\rangle$ at the time t_1 of the nth eigenfrequency has been evaluated by the moving time average, Rodrigues-Gomes [9]

$$\langle\omega_n(t)\rangle = \frac{1}{T_a}\int_{t-\frac{T_a}{2}}^{t+\frac{T_a}{2}} \omega_n(\tau)d\tau \ , \quad T_a = 2.4\frac{2\pi}{\omega_1(0)} \tag{1}$$

where T_a is the length of the averaging window.

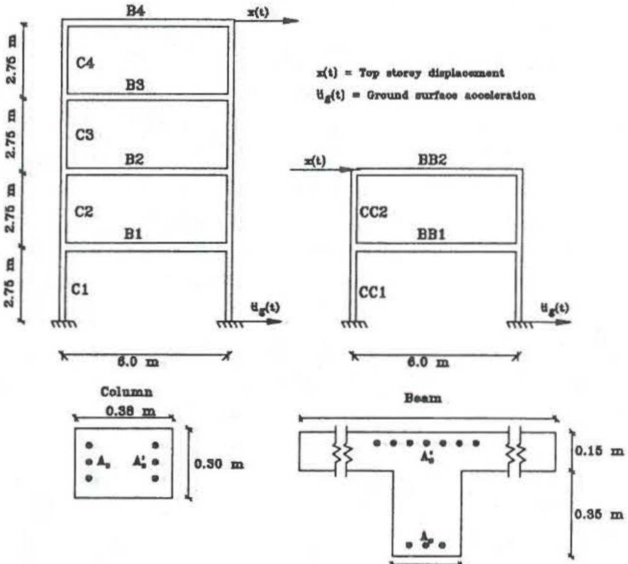

Figure 1: *Linear-elastic finite element models of reinforced concrete frames.*

The smoothed value of the fundamental frequency was used by DiPasquale et al. [2] used to define a global damage index δ_M called the maximum softening damage index. It is defined as the ratio between the undamaged fundamental eigenperiod T_0 and the maximum fundamental period $T_M = \max_{t\in[0,\infty[}\{T(t)\}$; of the softening system during a forced vibration event

$$\delta_M = 1 - \frac{T_0}{T_M} \tag{2}$$

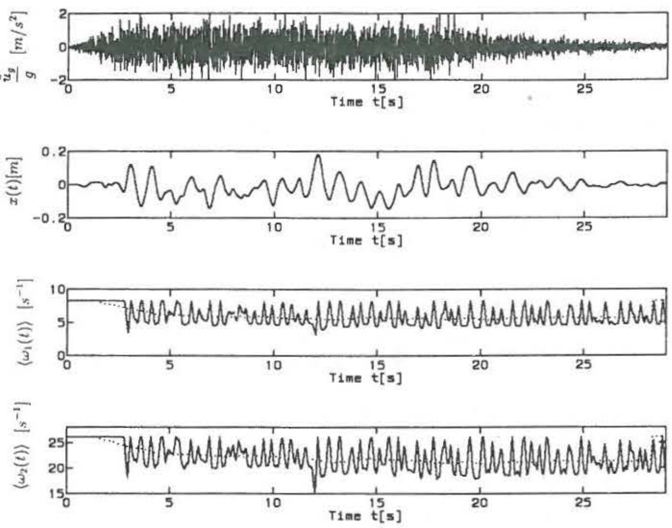

Figure 2: *4-storey frame. Realizations of the base acceleration, the top storey displacement and instantaneous and smoothed circular eigenfrequencies.*

The various local damage measures presented in this paper may be considered as extensions of these principes to the various substructures.

3 Localization Procedure

Assume that the structure in the initial undamaged state, where the structure behaves linearly elastic, is modelled by a finite element model providing the mass matrix $\mathbf{M}$ and the stiffness matrix $\mathbf{K}_0$. Then the initial circular eigenfrequencies $\omega_{i,0}$ and eigenmodes $\mathbf{\Phi}_{i,0}$ of the undamaged structure are obtained from the homogeneous linear equation

$$(\mathbf{K}_0 - \omega_{i,0}^2 \mathbf{M})\mathbf{\Phi}_{i,0} = \mathbf{0} \tag{3}$$

The damage localization is based on a sequence of substructurings in which the damage in each substructure is sequentially estimated. Initially, the structure is divided into two substructures labelled 1 and 2 as shown in figure 3a. Then

$$\mathbf{K}_0 = \mathbf{K}_{1,0}^{(1)} + \mathbf{K}_{2,0}^{(1)} \tag{4}$$

where $\mathbf{K}_{1,0}^{(1)}$ and $\mathbf{K}_{2,0}^{(1)}$ signify the global stiffness matrices of substructures 1 and 2. Although $\mathbf{K}_0$ is positive definite, its constituents $\mathbf{K}_{1,0}^{(1)}$ and $\mathbf{K}_{2,0}^{(1)}$ are both positive semi-definite, i.e. they contain a large number of zero components corresponding to the global positions of the extracted substructure. The subscripts 1 and 2 refer to substructures 1 and 2, and the subscript 0 refers to the initial state. The superscript (1) refers to the 1st time of substructuring.

Next, a stiffness matrix $\mathbf{K}_e(t)$ for the equivalent linear structure can be defined in the following way.

$$\mathbf{K}_e^{(1)}(t) = \left(1-\delta_1^{(1)}(t)\right)^2 \mathbf{K}_{1,0}^{(1)} + \left(1-\delta_2^{(1)}(t)\right)^2 \mathbf{K}_{2,0}^{(1)} \tag{5}$$

$\delta_1^{(1)}(t)$ and $\delta_2^{(1)}(t)$ signify the damage indicators for substructures 1 and 2, respectively. These may be interpreted as measures of the averaged stiffness loss in the substructure.

Next, $\delta_1^{(1)}$ and $\delta_2^{(1)}$ are identified, so $\mathbf{K}_e^{(1)}(t)$ as given by (5) provides the measured smoothed circular eigenfrequencies, $\langle\omega_1(t)\rangle$ and $\langle\omega_2(t)\rangle$, i.e.

$$\left(\sum_{j=1}^{2}\left(1-\delta_j^{(1)}(t)\right)^2 \mathbf{K}_{j,0}^{(1)} - \langle\omega_i(t)\rangle^2 \mathbf{M}\right)\mathbf{\Phi}_i(t) = \mathbf{0} \tag{6}$$

where $\mathbf{\Phi}_i(t)$ are the eigenmodes of the equivalent time-varying linear system.

The time-varying equivalent linear stiffness matrix of substructure 1 is then estimated as $\left(1-\delta_1^{(1)}(t)\right)^2 \mathbf{K}_{1,0}^{(1)}$. Next, the previously labelled substructure 2 can be divided into two new substructures, again labelled 1 and 2 as shown in figure 3b. Then a new stiffness matrix of the equivalent linear structure can be written on the form

$$\mathbf{K}_e(t) = \left(1-\delta_1^{(1)}(t)\right)^2 \mathbf{K}_{1,0}^{(1)} + \left(1-\delta_1^{(2)}(t)\right)^2 \mathbf{K}_{1,0}^{(2)} + \left(1-\delta_2^{(2)}(t)\right)^2 \mathbf{K}_{2,0}^{(2)} \tag{7}$$

where

$$\mathbf{K}_{2,0}^{(1)} = \mathbf{K}_{1,0}^{(2)} + \mathbf{K}_{2,0}^{(2)} \tag{8}$$

It should here be noted, that when only one substructure is used, the corresponding damage $\delta_1(t)$ measure is equivalent to the global softening obtained from eq. (2).

Since $\delta_1^{(1)}(t)$ is known, $\delta_1^{(2)}(t)$ and $\delta_2^{(2)}(t)$ can be estimated, inserting (7) into (6). From a new system identification, $\delta_1^{(2)}(t)$ and $\delta_2^{(2)}(t)$ are then obtained.

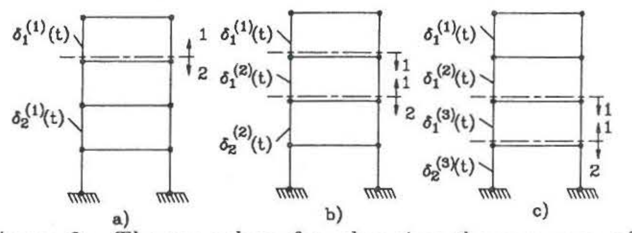

Figure 3: *The procedure for changing the sequence of substructuring.*

The procedure of dividing the previously labelled substructure 2 into 2 new substructures can be repeated further. Assume that this procedure has been performed

i times, where eq. (5) corresponds to $i = 1$ and eq. (7) to $i = 2$. Then the stiffness matrix of the equivalent linear system can be written as.

$$\mathbf{K}_e^{(i)}(t) = \sum_{j=1}^{i-1} \left(1 - \delta_1^{(j)}(t)\right)^2 \mathbf{K}_{1,0}^{(j)} + \left(1 - \delta_1^{(i)}(t)\right)^2 \mathbf{K}_{1,0}^{(i)}$$

$$+ \left(1 - \delta_2^{(i)}(t)\right)^2 \mathbf{K}_{2,0}^{(i)} \tag{9}$$

In eq. (9) $\delta_1^{(1)}(t), \ldots, \delta_1^{(i-1)}(t)$ are known from previous identifications. $\delta_1^{(i)}(t)$ and $\delta_2^{(i)}(t)$ can be identified by inserting (9) into (6). Below, all the contributions to the stiffness from previous iterations, i.e. the summation $\sum_{j=1}^{i-1} \left(1 - \delta_1^{(j)}(t)\right)^2 \mathbf{K}_{1,0}^{(j)}$ will be referred to as $\mathbf{K}_{0,0}^{(i)}$ for convenience of notation.

By applying the above procedure, $\delta_1^{(i)}$ provides a measure of the average damage of each storey. If further localization within a given storey needs to be performed, it can in principle be done by fixing the damage at all other stories with the local damages determined previously. The storey to be investigated can then be divided into two new substructures and new local damages can be determined. When using the method it should be kept in mind that symmetrically placed elements in a symmetric structure will cause the same change in eigenfrequencies, and the localization is therefore limited to one of two possibilities.

4 Identification of Local Damages

In this section, an iterative method used for identification of the local damages $\delta_1^{(i)}(t)$, $\delta_2^{(i)}(t)$, at the ith level of substructuring is described.

Initially, the eigenvalue problem (3) is solved by means of a subspace iteration yielding the two lowest eigenfrequencies $\omega_{1,0}$, $\omega_{2,0}$ and the corresponding mode shapes $\mathbf{\Phi}_{1,0}$ and $\mathbf{\Phi}_{2,0}$ of the undamaged structure. The values of $\delta_1^{(i)}(t)$ and $\delta_2^{(i)}(t)$ at the nth step of the iteration process are designated $\delta_{1,n}^{(i)}(t)$, $\delta_{2,n}^{(i)}(t)$, respectively. These are then determined from the Rayleigh fraction

$$\langle\omega_j(t)\rangle^2 = \frac{\mathbf{\Phi}_{j,n-1}^T \mathbf{K}_e\left(\delta_{1,n}^{(i)}, \delta_{2,n}^{(i)}, t\right) \mathbf{\Phi}_{j,n-1}}{\mathbf{\Phi}_{j,n-1}^T \mathbf{M} \mathbf{\Phi}_{j,n-1}}, \quad n = 1, 2, .. \tag{10}$$

where $\mathbf{\Phi}_{j,n-1} = \mathbf{\Phi}_{j,n-1}(t)$ are the eigenmodes at the $(n-1)$th step of iteration, i.e. corresponding to using the stiffness matrix $\mathbf{K}_e(\delta_{1,n-1}^{(i)}(t), \delta_{2,n-1}^{(i)}(t))$ in (6). At the first step for $n = 1$, the undamaged eigenmodes $\mathbf{\Phi}_{j,0}$ are applied. Insertion of $\mathbf{K}_e(\delta_{1,n}^{(i)}(t), \delta_{2,n}^{(i)}(t))$ given by (9) into (10) provides the following two linear equations in $\left(1 - \delta_{1,n}^{(i)}(t)\right)^2$ and $\left(1 - \delta_{2,n}^{(i)}(t)\right)^2$ for the determination of the damage measures of nth iteration step.

$$\langle\omega_j(t)\rangle^2 = \frac{\mathbf{\Phi}_{j,n-1}^T \mathbf{K}_{1,0}^{(i)} \left(1 - \delta_{1,n}^{(i)}(t)\right)^2 \mathbf{\Phi}_{j,n-1}}{\mathbf{\Phi}_{j,n-1}^T \mathbf{M} \mathbf{\Phi}_{j,n-1}}$$

$$+ \frac{\mathbf{\Phi}_{j,n-1}^T \mathbf{K}_{2,0}^{(i)} \left(1 - \delta_{2,n}^{(i)}(t)\right)^2 \mathbf{\Phi}_{j,n-1}}{\mathbf{\Phi}_{j,n-1}^T \mathbf{M} \mathbf{\Phi}_{j,n-1}}$$

$$+ \frac{\mathbf{\Phi}_{j,n-1}^T \mathbf{K}_{0,0}^{(i)} \mathbf{\Phi}_{j,n-1}}{\mathbf{\Phi}_{j,n-1}^T \mathbf{M} \mathbf{\Phi}_{j,n-1}}, \quad j = 1, 2 \tag{11}$$

From the determined values of the local damages $\delta_{1,n}^{(i)}(t)$, $\delta_{2,n}^{(i)}(t)$, a new equivalent stiffness matrix can be calculated and new eigenmodes $\mathbf{\Phi}_{1,n}$, $\mathbf{\Phi}_{2,n}$ can be found from

$$\left(\mathbf{K}_e^{(i)}(\delta_{1,n}^{(i)}(t), \delta_{2,n}^{(i)}(t)) - \langle\omega_i(t)\rangle^2 \mathbf{M}_0\right) \mathbf{\Phi}_{i,n} = \mathbf{0} \tag{12}$$

This procedure eq. (10) to eq. (12) is looped in each substructuring until no changes occur in the local damage, i.e. $|\delta_{1,n}^{(i)} - \delta_{1,n-1}^{(i)}| + |\delta_{2,n}^{(i)} - \delta_{2,n-1}^{(i)}| < \epsilon$, where ϵ is a tolerance of the magnitude 10^{-5}. During the iteration process it is checked whether $(1 - \delta_{j,n}^{(i)(t)})^2$ becomes negative or larger than 1. If so, the ajustments $(1 - \delta_{j,n}^{(i)})^2 = 0$ or $(1 - \delta_{j,n}^{(i)})^2 = 1$ are imposed.

5 The programme SARCOF

All data in this paper is generated by the non-linear finite element programme SARCOF, Mørk [6] which is an acronym for "Stochastic Analysis of Reinforced COncrete Frames". In the programme special emphasis is put on analysis of stiffness degradation due to severe plastic deformations. Furthermore, the finite length of plastic zones is taken into account upon controlling the plasticity at the end sections and at three internal cross-sections of the member.

As a result, a piecewise linear estimate of the slowly varying bending stiffness $EI(x,t)$ is obtained. An equivalent homogenous bending stiffness $\langle EI(t)\rangle$ in the element with the length l can then be determined from averaging the flexibilities

$$\frac{l}{\langle EI(t)\rangle} = \int_0^l \frac{dx}{EI(x,t)} \tag{13}$$

Based on this averaged value of the bending stiffness, a local damage $\delta(t)$ of the element can be determined from

$$\langle EI(t)\rangle = (1 - \delta(t))^2 EI_0 \quad \Rightarrow \quad \delta(t) = 1 - \sqrt{\frac{\langle EI(t)\rangle}{EI_0}} \tag{14}$$

where EI_0 is the constant bending stiffness of the initially uncracked beam.

The determination of this local damage for each element has been implemented in the SARCOF programme and is used for comparison with the local damages evaluated by the present method.

6 Numerical examples

Two structures of different complexity are considered in this section. A 1-bay, 2-storey and a 1-bay, 4-storey RC-frame with identical beams and columns. The method is tested for each of the two structures with 3 different earthquake intensities.

Eksample 1:

The first structure considered is the 2-storey 1-bay framed structure shown in figure 1.
The modulus of elasticity of reinforcement bars is $E = 2.1 \cdot 10^{11}$Pa, the mass density of the concrete is $\rho_c = 2500$kg/m^3. All columns and beams are symmetrically reinforced.

The resisting parameters of the cross-sections are given in table 1. The calculated eigenfrequencies of the undamaged building were $\omega_{1,0} = 16.483\text{s}^{-1}$, $\omega_{2,0} = 52.947\text{s}^{-1}$. A damping ratio of 0.05 is assumed in all modes.

	A'_s [10^{-3}m^2]	A_s [10^{-3}m^2]	I [10^{-3}m^4]	E_c [10^{10}Pa]
Beam	1.64	1.14	1.1	5.0
Column	1.64	1.64	0.5	5.0

Table 1: *Characteristics of cross-sections.*

The earthquake applied is characterized by the following parameters. Time $t_1 = 3$s for start of maximum intensity, duration of maximum intensity $t_0 = 15$s, decay ratio for intensity $c = 0.2\text{s}^{-1}$, circular eigenfrequency of Kanai-Tajimi filter $\omega_0 = 15\text{s}^{-1}$ and damping $\zeta_0 = 0.65$, Mørk [6].

The structure considered is exposed to the earthquake shown in figure 2a with an EQ intensity of $\beta_0 = 0.8\text{m/s}^{\frac{3}{2}}$. Using the smoothing by the eq. (1), the development of the two lowest eigenfrequencies are as illustrated by a solid line in figure 5. It is seen that the maximum softening is $1 - \frac{9.0}{16.4} = 0.45$.

By applying the time series of the eigenfrequencies to the present method, the local damages illustrated in figure 4 are obtained. These are compared to the "correct" local damages obtained directly from SARCOF.

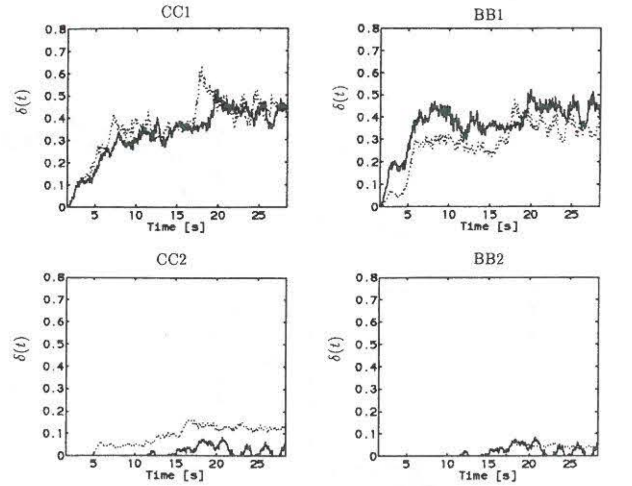

Figure 4: *2-storey frame. Estimated local damages with $\beta_0 = 0.8m/s^{\frac{3}{2}}$. [——]: Present method, [···]: SARCOF.*

It is seen that the local damages determined by the present method is very close to the "correct" values. The local damage in columns and beams of the lower storey is almost exactly predicted whereas the local damage in the upper storey columns is slightly underestimated. In figure 5, it is seen that the frequencies reproduced by the present method are very similar to the measured frequencies.

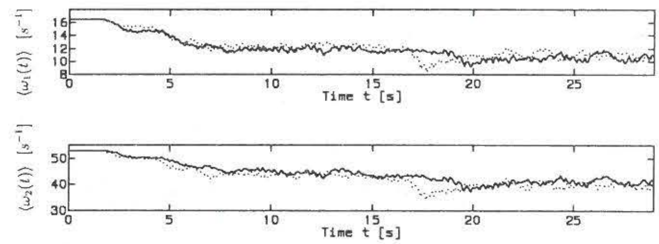

Figure 5: *2-storey frame. Comparison of reproduced eigenfrequencies. [——]: Measured eigenfrequencies, [···]: SARCOF. $\beta_0 = 0.8m/s^{3/2}$.*

In figures 6 and 7 the corresponding results are shown for the cases, where $\beta_0 = 0.6\text{m/s}^{3/2}$ and $\beta_0 = 0.4\text{m/s}^{3/2}$ respectively.
As the intensity of the earthquake decreases it is seen that the uncertainty of the localization of damage increases. For very low earthquake intensities as in figure 7 it is seen that the method actually predicts damage in a column which is undamaged. But in all cases it is seen that the elements with the highest degree of damage are pin-pointed correctly. At low earthquake intensities it is seen that the magnitude is quite poorly predicted.

Eksample 2:

Next, the 4-storey 1-bay framed structure shown in fi-

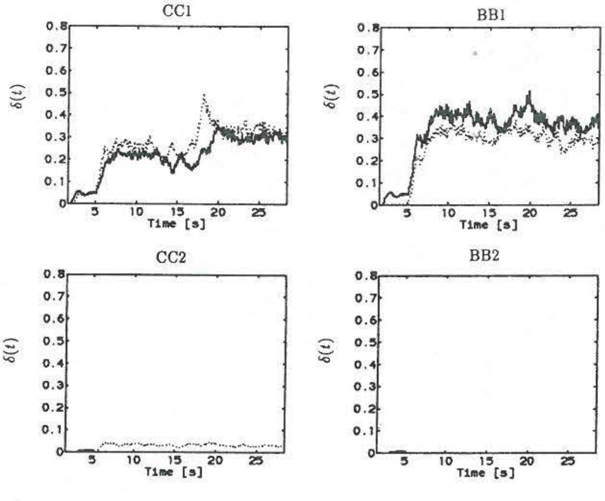

Figure 6: *2-storey frame. Estimated local damages with* $\beta_0 = 0.6m/s^{3/2}$. *[—]: Present method, [···]: SARCOF.*

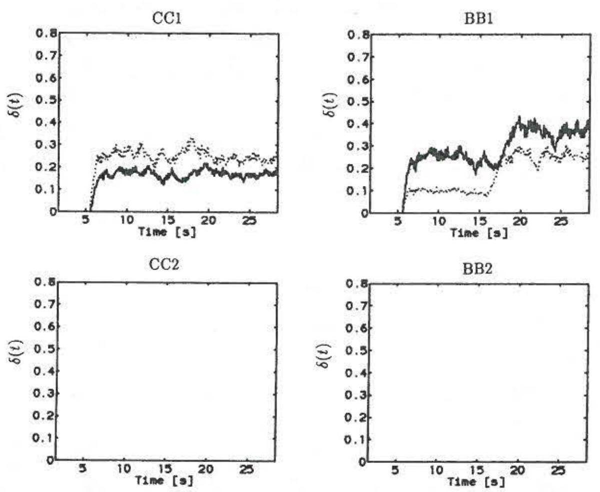

Figure 7: *2-storey frame. Estimated local damages with* $\beta_0 = 0.4m/s^{3/2}$. *[—]: Present method, [···]: SARCOF.*

gure 1 is considered. All beams and columns are identical to the ones used in example 1. Using the smoothing by the eq. (1) the development of the two lowest eigenfrequencies is as illustrated by the solid line in figure 9. It is seen that the maximum softening is $1 - \frac{5.40}{8.26} = 0.35$. The reason is of course that the eigenfrequency has been further removed from the dominating frequencies of the excitation.

The local damages of the elements are illustrated in figure 8.

It is seen that also in this case the local damages determined by the present method are very close to the "correct" values and that severely damaged elements are pin-pointed very precisely.

In figure 10 and 11 the corresponding results are shown for the cases, where $\beta_0 = 0.4\text{m/s}^{3/2}$ and $\beta_0 = 0.2\text{m/s}^{3/2}$,

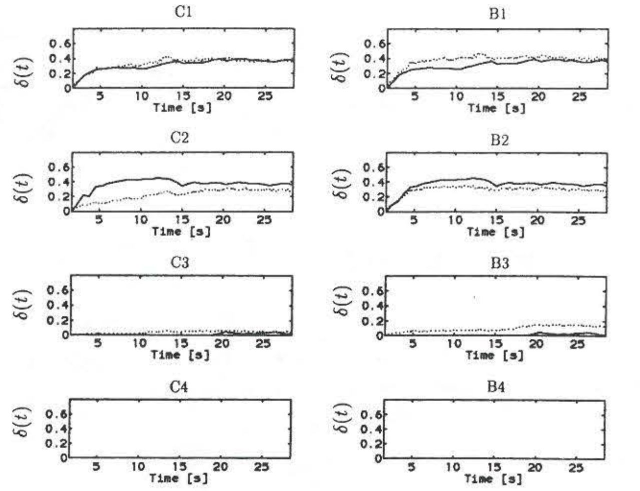

Figure 8: *4-storey frame. Estimated local damages with* $\beta_0 = 0.6m/s^{\frac{3}{2}}$. *[—]: Present method, [···]: SARCOF.*

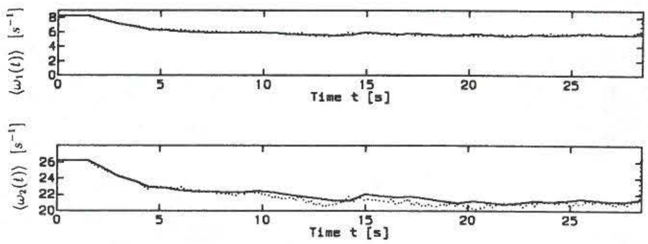

Figure 9: *4-storey frame. Comparison of reproduced eigenfrequencies by the present method [– · – · –], the measured eigenfrequencies [—] and eigenfrequencies determined from local damages calculated by SARCOF [···].* $\beta_0 = 0.6m/s^{3/2}$.

respectively.

It is seen from the figures 10-11 that, as the intensity of the earthquake decreases, the uncertainty of the localization of damage increases. For very low earthquake intensities, as in figure 11, it is seen that the method actually predicts damage in a column which is undamaged.

7 Conclusions

A method for localization and quantification of damage in structural systems is tested for RC-structures subject to earthquakes. It is found that the method works very well when the structure is severely damaged. At lower damage levels the method yields slightly higher uncertainty, but the localization is still in the correct parts of the structure. It should be noted that in the cases where the method predicts damage in undamaged elements, the error is limited to neighbouring elements which means that the damage is predicted to be located in the area close to the actual damage location. The results from the two numerical examples indicate that the uncertainty of

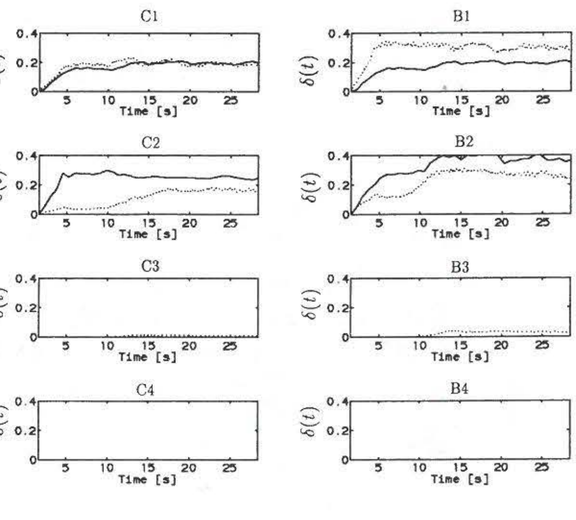

Figure 10: *4-storey frame. Estimated local damages with* $\beta_0 = 0.4 m/s^{3/2}$. *[—]: Present method, [···]: SARCOF.*

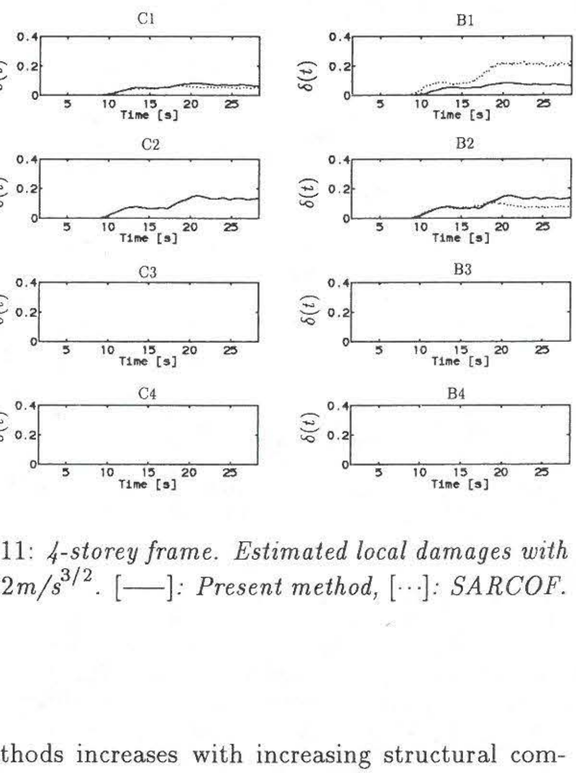

Figure 11: *4-storey frame. Estimated local damages with* $\beta_0 = 0.2 m/s^{3/2}$. *[—]: Present method, [···]: SARCOF.*

the methods increases with increasing structural complexity, which can be explained by the fact that more and more parameters are required to be estimated from the same amount of data. The main conclusions to be drawn from the works connected with this paper is that the investigated method is very suitable for localization and quantification of damage in structural elements in simple RC-structures. For more complex structural systems the damage localization has to be confined to a corresponding number of substructures. Damage localization at element level requires more available information, in this case e.g. measurement of mode shapes etc.

8 Acknowledgement

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