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STRUCTURAL RELIABILITY THEORY
PAPER NO. 149

To be presented at the Localised Damage Conference, Fukuoka, Japan, June 1996

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Abstract

This paper deals with the estimation of the damage location of severely damaged Reinforced Concrete (RC) structures excited by earthquakes. It is assumed that the building is instrumented with a sensor measuring the earthquake acceleration signal at ground surface and a sensor measuring only a single output response such as the top storey displacement. Based on these measurements it is assumed that the two lowest smoothed averaged circular eigenfrequencies can be estimated. To get a damage measure in terms of stiffness loss, a relation between stiffness loss and the time variation of the smoothed eigenfrequencies must be established. The idea of the method is to use a linear finite element model of the undamaged structure and then repeatingly partitioning it into a number of substructures. For each step of this process a damage indicator, defined as the average relative reduction of stiffness, is determined for a certain substructure. The overall example in the paper is a 4-storey 1-bay RC-structure. This model is artificially damaged in a specific member and the proposed method is verified in this case. It should be noted that the data used in this paper is completely artificial and is kept free of all polution sources, i.e. uncertainties in measurements, noise, bias and uncertainty of the identification procedure, etc. An investigation of the influence of polluted data is considered in another paper.

Keywords: Damage, Localization, RC-structure, System identification.

1 Introduction

During the last decade assessment of damage in structures has attracted much interest from various researchers. Until now the focus has mainly been 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 ($< 5 - 20\%$). However, this is not the case for RC-structures during e.g. an earthquake, where the changes in the modal parameters are considerable (often > 100%). 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.

The methods published in the literature can generally be divided into 3 categories. The principle in the first category of methods, which includes the works of Cawley and Adams [2], Stubbs and Osegueda [20] and Penny et al., [16] is to use measured changes in modal quantities (eigenfrequencies/eigenmodes) and then compare these with changes in modal parameters obtained from a sensitivity analysis of a finite element model based on linear expansion from the undamaged state. The damage is then located where the best match is found. This method requires a high number of eigenfrequencies to be estimated (> 5) which is why it is not suited for seismicly excited RC-structures. Most of these methods are based on comparison of eigenfrequencies obtained in an arbitrary number of different damage scenarios obtained from a finite element analysis and the measured changes. In the second category the principle is to carry out a mathematical fitting of the relation between damage location/size and the measured changes in modal parameters without any physical information. Until now methods in this category have most succesfully been solved by use of static multilayer neural networks trained with back-propagation as in e.g. Kirkegaard and Rytter [8], Skjærbæk and Asmussen [18] and Szewczyk and Hajela [21]. To estimate the parameters in the network (network training) sensitivities obtained from analytical models are normally used. The success has been of varying quality mainly due to lack of physical knowledge when designing the network and to generation of local minima in the network giving unstable solutions. In the third category the inverse problem, solving structural parameters (mass, stiffness or damping) from measured modal parameters is dealt with. This is perhaps the most promising group of methods and has been dealt with in several papers as e.g. Park et al. [15], Pandey and Biswas [14] and Hassiotis and Jeong [6]. The method of Park et al. [15] is a stiffness error matrix method based on the difference between analytical stiffness and measured modal properties. For large changes in stiffness it was found effective, and the main objection to this method is that a relatively large number (> 5) of eigenfrequencies need to be measured. Pandey and Biswas [14] set up a relation between changes in modal parameters and changes in flexibility. They argued that the estimate of the flexibility matrix converged very fast with the number of eigenfrequencies so the flexibility can be estimated from a few eigenfrequencies. Hassiotis and Jeong [6] used the same method based on changes in structural stiffness. From a strict mathematical point of view the number of estimated damage indicators has to be the same as the number of estimated eigenfrequencies. However, in e.g. Hassiotis and Jeong [6] the problem with a larger number of damage indicators than frequency estimates was tackled by an optimality criterion.

When it comes to severely damaged RC-structures the development of methods for assessment of damage has mainly been concentrated on global damage indicators which are measures of the global state of damage in a structure. In contrast to damage assessment in steel structures, where first order perturbation is a good approximation, the damage assessment in RC-structures requires a more careful handling of the nonlinear terms due to large changes in frequencies. This is probably the reason why development of methods for damage assessment of RC-structures from experimental measurements until now has been very limited. Assessment of damage in RC-structures was originally related to changes in modal parameters by DiPasquale and Çakmak [4] who proposed the maximum relative reduction of the first smoothed eigenfrequency as a global damage indicator. This so-called Maximum Softening Damage Index (MSDI) is a weighted average of stiffness degradation due to plastic deformations which occur during e.g. extreme earthquake excitations and the stiffness degradation due to damage. Correlations between actual damage levels and damage levels computed with the MSDI have been demonstrated through seismic assessment of actual strong motion records from medium rise RC-structures subjected to the 1971 San Fernando earthquake, DiPasquale and Çakmak [5]. This indicator has been further investigated in several papers as e.g. Rodriguez-Gomes [17], Nielsen et al. [12], Iwankiewicz et al. [7], Köylüoğlu et al. [9] and Nielsen et al. [13]. Köylüoğlu et al. [9] proposed a method for localization of damage by fitting an MDOF hysteretic oscillator to measured response at m measuring points along an RC-building and the ground motions. By this method an estimate of the damage between each measuring point can be found. The major problem with this method is the need for many measuring points which will require heavy instrumentation of the building.

The objective of this paper is to present a method for localization of damage in RC-structures, where only displacement response at the top storey and the ground acceleration is measured. The method is based on estimation of the smoothed values of the two lowest circular eigenfrequencies of the structure from measured response signals. Normally, these modes are the only ones excited during an earthquake, so information of higher modes is not likely to be available. The damage of a certain magnitude will be localized by a sequence of equivalent linear systems, which are formulated by means of a linear finite element model of the structure. The method is demonstrated in an example where a single beam has been artificially damaged. The smoothed values of the eigenfrequencies are also generated artificially to make sure that the data are free from estimation errors.

2 Description of Localization 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 [4], Köylüoğlu et al. [9], time-averaging FFT, Mullen et al. [11] or discrete wavelet transforms, Micaletti et al. [10]. These methods give somewhat different results, since they depend highly 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.

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 $\mathbf{K}_0$. The linear undamaged eigenvibrations of the structure can then be described by the following system of differential equations

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{K}_0\mathbf{x} = \mathbf{0} \tag{1}$$

where $\mathbf{x}$ is the displacement vector.

The initial circular eigenfrequencies $\omega_{i,0}$ and eigenmodes $\mathbf{\Phi}_{i,0}$ of the undamaged structure are obtained from the homogeneous linear equation

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

The damage localization is based on a sequence of sub-structurings 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 1a. Then

$$\mathbf{K}_0 = \mathbf{K}_{1,0}^{(1)} + \mathbf{K}_{2,0}^{(1)} \tag{3}$$

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 constitutives $\mathbf{K}_{1,0}^{(1)}$ 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 refers 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.

$$\mathbf{K}_e^{(1)}(t) = \left(1 - \delta_1^{(1)}(t)\right)^2 \mathbf{K}_{1,0}^{(1)} + \left(1 - \delta_2^{(1)}(t)\right)^2 \mathbf{K}_{2,0}^{(1)} \tag{4}$$

$\delta_1^{(1)}(t)$ 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. The global softening, $\delta(t)$, of the structure is obtained using the equivalent linear stiffness matrix $\mathbf{K}_e(t) = (1-\delta(t))^2\mathbf{K}_0$, and the maximum softening damage index is the maximum value of $\delta(t)$. Hence, the maximum values of $\delta_1^1(t)$ and $\delta_2^1(t)$ can be interpreted as maximum softening damage indices for the substructures 1 and 2, respectively.

Next, $\delta_1^{(1)}$ and $\delta_2^{(1)}$ are identified, so $\mathbf{K}_e^{(1)}(t)$ as given by (4) provides the measured smoothed circular eigenfrequencies, $\langle\omega_1(t)\rangle$ and $\langle\omega_2(t)\rangle$, i.e.

$$\left(\mathbf{K}_e(t) - \langle\omega_i(t)\rangle^2\mathbf{M}\right)\mathbf{\Phi}_{i,0} = \mathbf{0} \tag{5}$$

or

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

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 1b. 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

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

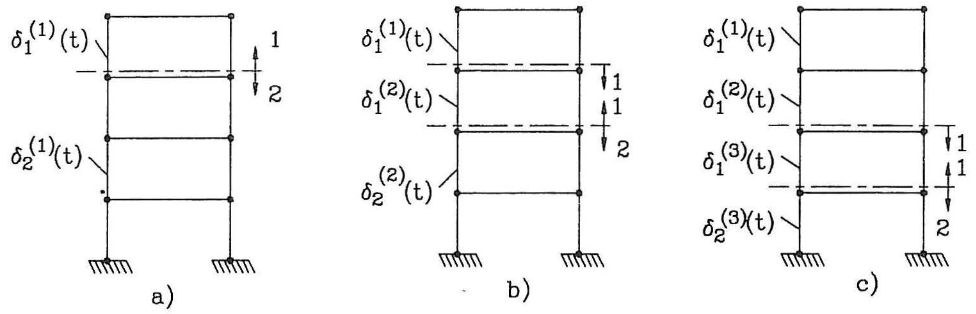

Figure 1: *The procedure for changing the sequence of substructuring.*

Since $\delta_1^{(1)}(t)$ is known, $\delta_1^{(2)}(t)$ and $\delta_2^{(2)}(t)$ can be estimated, inserting (7) into (5). From a new system identification, $\delta_1^{(2)}(t)$ and $\delta_2^{(2)}(t)$ are obtained.
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. (4) 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 (5). All the contributions to the stiffness from previous iterations, i.e. the summation $\sum_{j=1}^{i-1} \left(1 - \delta_1^{(j)}(t)\right)^2 \mathbf{K}_{1,0}^{(j)}$ is 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 softenings determined previously. The storey to be investigated can then be divided into two new substructures and new softenings can be determined.

In case of symmetric structures the method has one main theoretical limitation, where the method is only capable of localizing the damage to one of two possibilities. This limitation is illustrated in figure 2, where it is seen that the damage scenarios illustrated in the figure to the left will give exactly the same changes in eigenfrequencies as the scenarios shown at the figure to the right. From measured eigenfrequencies alone it will not be possible to separate these scenarios.

3 Algorithm for Identification of Softenings

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

Initially, the eigenvalue problem (2) 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

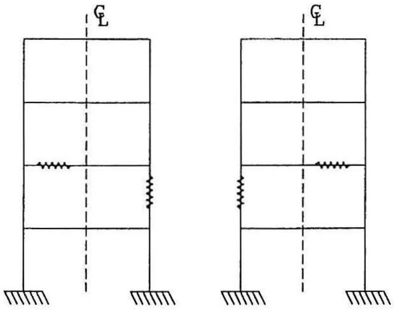

Figure 2: *Damage scenarios giving the same change in eigenfrequencies.*

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

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

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 (5). 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.

$$\langle\omega_j(t)\rangle^2 = \frac{\mathbf{\Phi}_{j,n-1}^T \mathbf{K}_{1,0}^{(i)} \left(1 - \delta_{1,n}^{(i)}(t)\right)^2 \mathbf{\Phi}_{j,n-1} + \mathbf{\Phi}_{j,n-1}^T \mathbf{K}_{2,0}^{(i)} \left(1 - \delta_{2,n}^{(i)}(t)\right)^2 \mathbf{\Phi}_{j,n-1}}{\mathbf{\Phi}_{j,n-1}^T \mathbf{M} \mathbf{\Phi}_{j,n-1}}$$
$$+ \frac{\mathbf{\Phi}_{j,n-1}^T \mathbf{K}_{0,0}^{(i)} \mathbf{\Phi}_{j,n-1}}{\mathbf{\Phi}_{j,n-1}^T \mathbf{M} \mathbf{\Phi}_{j,n-1}}, \quad j = 1, 2 \tag{11}$$

From the determined values of the softenings $\delta_{1,n}^{(i)}(t)$, $\delta_{2,n}^{(i)}(t)$ 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 softenings, 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.

In principle the softening could be determined at all time steps, so the development in damage could be followed. In practice it is sufficient to analyse the structure in a much shorter interval after the arrival of the peak acceleration where the maximum reduction of the eigenfrequencies are present.

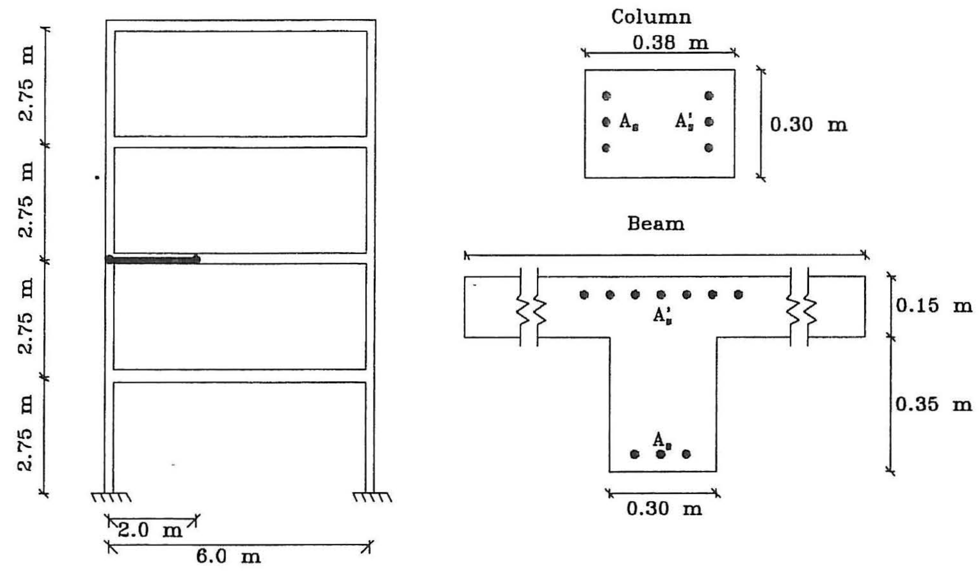

Figure 3: *Linear-elastic finite element model of reinforced concrete frame.*

4 Numerical Example

The structure considered in this paper is a 4-storey 1-bay framed structure as illustrated in figure 3, Nielsen et al. [13].

The modulus of elasticity of reinforcement bars is $E = 2.1 \cdot 10^{11}\text{Pa}$, the mass density of the concrete is $\rho_c = 2500\text{kg/m}^3$. All columns and beams are symmetrically reinforced.

The resisting parameters of the cross-sections are given in table 1. The calculated eigenfrequencies of the building were $\omega_{1,0} = 8.26\text{s}^{-1}$, $\omega_{2,0} = 26.22\text{s}^{-1}$.

	A'_s $[10^{-3}\text{m}^2]$	A_s $[10^{-3}\text{m}^2]$	I $[10^{-3}\text{m}^4]$	E_c $[10^{10}\text{Pa}]$
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 considered damage scenario assumes the 2nd storey beam to be damaged in the outermost 2 m, whereas the rest of the structure is undamaged. The damage is assumed to increase linearily from $\delta_1 = 0$ at $t = 0$ to $\delta_1 = 0.40$ at $t = 20$ as illustrated in figure 4.

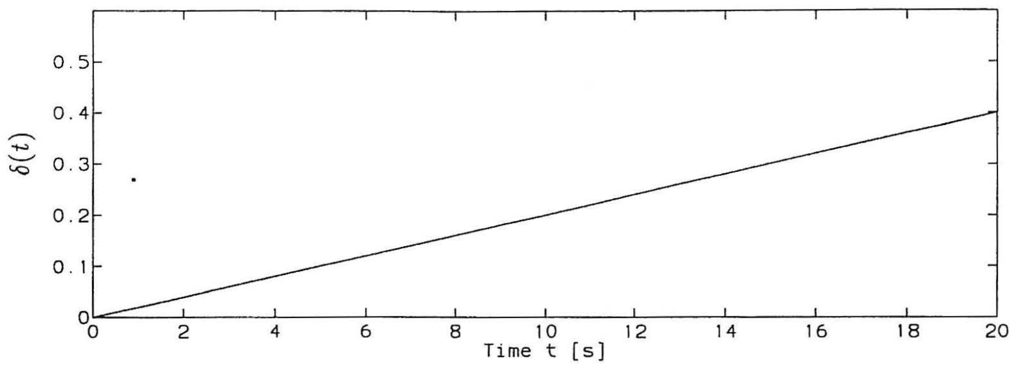

Figure 4: *The assumed development of damage in the outmosts part of the 2nd storey beam.*

The idea is to generate artificially time-varying smoothed circular eigenfrequencies $\langle\omega_i(t)\rangle$ corresponding to what would have been measured. According to this damage scenarios the damage is introduced in a FEM and an eigenvalue analysis is performed at each time step yielding the corresponding circular eigenfrequencies $\langle\omega_1(t)\rangle$ and $\langle\omega_2(t)\rangle$ as illustrated in figure 5.

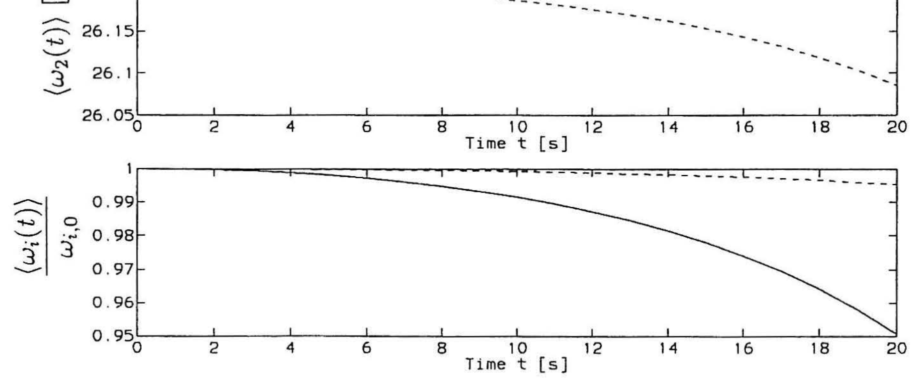

Figure 5: *The development in the two lowest eigenfrequencies using the damage development shown in figure 4 for beam no. 10a. a) 1st circular eigenfrequency $\omega_1(t)$, b) 2nd circular eigenfrequency $\omega_2(t)$ and c) $\omega_1(t)$, $\omega_2(t)$ normalized with respect to the undamaged value.*

It should be noted that this artificial example is merely presented to verify the method in an easily managable case rather than to give a realistic impression of the damage development in an actual RC-structure.

The sequence of substructuring, and the associated estimates of the softenings have been shown in figures 6 and 7. Figures 6c and 6d represent 2 alternative substructurings at the 3rd level. In figure 7c it is clearly seen that the damage must be present in the 2nd storey beam and figure 7d provides the final solution corresponding to the substructuring in figure 6d. Further localization into a part of the beam fails due to symmetry, which causes multiple solutions as explained earlier.

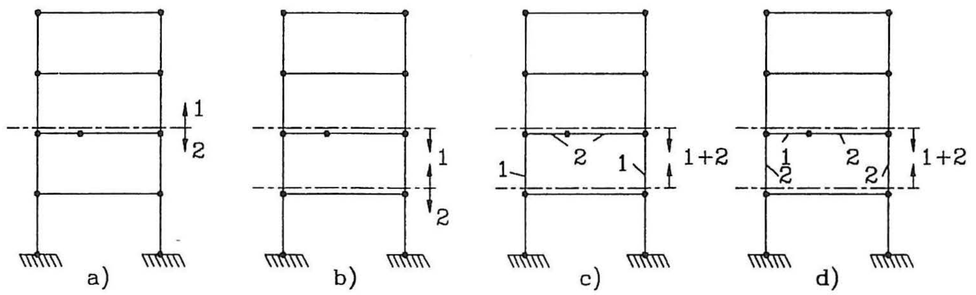

Figure 6: *Sequence of substructuring in search for damaged member.*

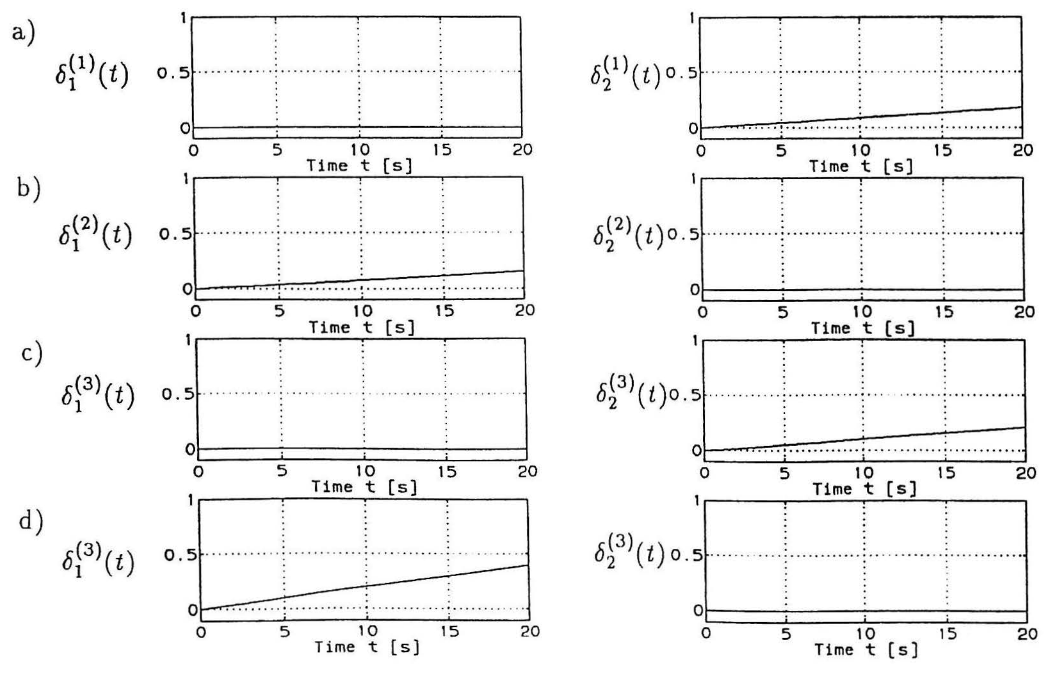

Figure 7: *Development in softenings for different partitionings.*

5 Conclusions

A method for localization and quantification of damage in structural systems is proposed. Special emphasis is put on severely damaged RC-structures, due to the heavy nonlinearity of this class of structures. It is found that the method is able to locate a damaged member in a RC-structure based on synthetic data where all noise polution sources have been eliminated. The localization capability of the method in the case of symmetric structures is limited to one of two possible locations of the damage. The main conclusion from the work done in connection with this paper is that the proposed method has potential for use as a damage localization method for RC-structures, at least if the measurements of $\langle\omega_1(t)\rangle$ and $\langle\omega_2(t)\rangle$ are accurate. Further investigations of the influence of noise in these measurements and the related identification capability is needed in order to assess the efficiency of the method.

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