Shaking Table Tests of Reinforced Concrete Frames

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ABSTRACT

The purpose of the paper is to present a series of shaking table experiments performed at the Structural Laboratory at Aalborg University, Denmark during the autumn of 1996 and to show some selected results from these experiments. The aim of the tests was to test methods for identification of time-varying systems and to verify various methods for damage assessment of reinforced concrete structures from strong motion measurements. In this study the maximum softening concept will be evaluated. In the paper the damage assessment obtained by this method is compared to visual damage assessment. The structures considered in the shaking table tests are 2-bay, 6-storey RC-frames in scale 1:5 with outer measures of 2.4 m in width and 3.3 m in height. The structures are subjected to a series of sequential earthquakes and after each earthquake the structure is visually inspected. The results of the work have revealed that the recursive vector ARMA model is suitable for modal identification of degrading reinforced concrete structures and the maximum softening damage index calculated from the obtained identification provides a valuable tool for assessment of the damage state of the structure.

INTRODUCTION

When civil engineering structures are subjected to sufficiently high dynamic loads it is well known that damage will occur somewhere in the structure. In RC-structures the damage normally starts as cracking developing into crushing of concrete and yielding of reinforcement. The damage can either be highly localized or more spread out in the structure. During an earthquake both types of damage may develop in the structure and there is a need for methods for localization of the damage. The traditional way of assessing damage in RC-structures is by visual inspection of the structure by measuring cracks, permanent deformations, etc. This is often very cumbersome or not possible, since panels and other walls covering beams and columns need to be removed. Furthermore, internal damage such as bond slippage can be very difficult to determine by visual inspection. However, a much more attractive method is measuring of the structural response at a given location of the structure. From these response time series, damage indicators based on e.g. changes in dynamic characteristics, accumulated dissipated energy, low cycle fatigue models, stiffness or flexibility changes etc. can be calculated. In the literature several methods for damage assessment from measured responses have been presented during the last 2 decades, see e.g. Banon et al. [1], DiPasquale et al. [2], Park et al. [5], Skjærbæk et al. [6] and Stephens et al. [7].

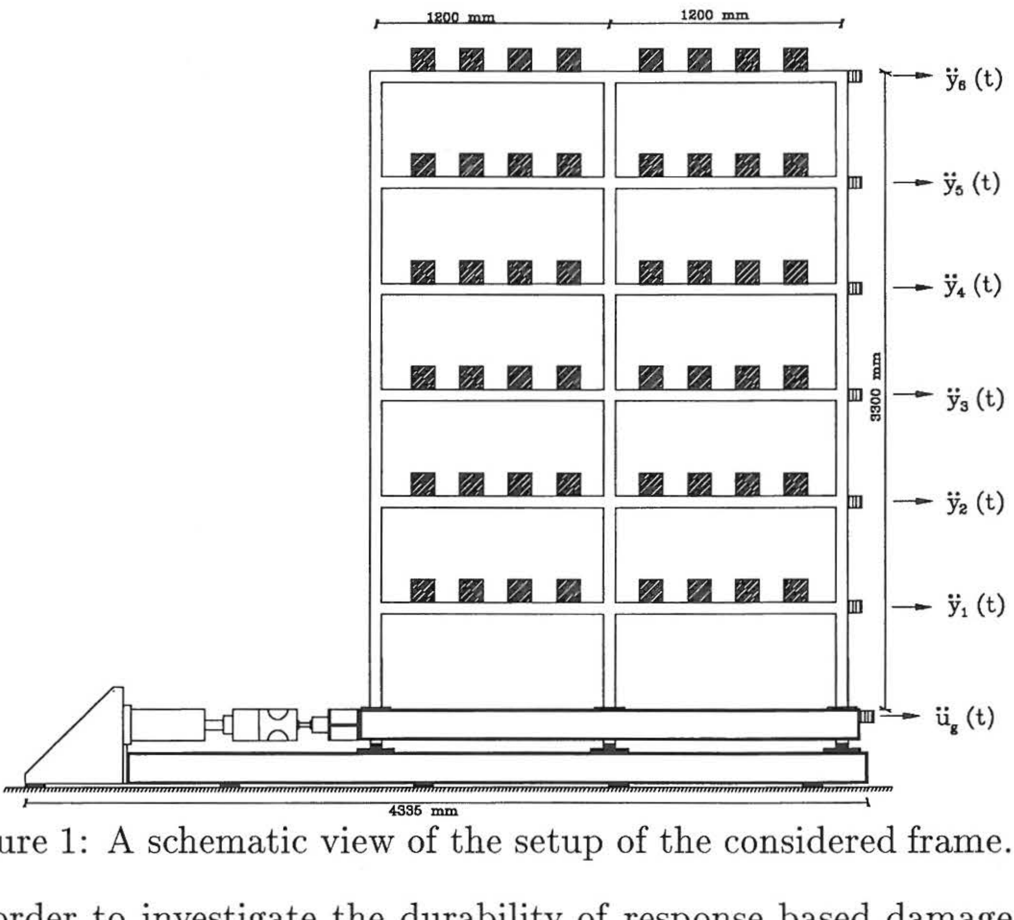

Figure 1: A schematic view of the setup of the considered frame.

In order to investigate the durability of response based damage assessment methods scaled model shaking table tests are needed. The purpose of the shaking table experiments performed on a series of 6-storey, 2-bay model test frames in scale 1:5 is therefore to provide data for verification and validation of these methods for non-destructive damage assessment of RC-frames based on one or more measured reponses of the structure. A schematic view of the test set-up is shown in figure 1.

However, the scope of this paper is limited to evaluate the maximum softening damage index as a damage indicator for the considered reinforced concrete frame.

THE TEST SERIES

All the 7 frames considered in the test series were constructed identically. The dimensions of the test frames are 2400 by 3300 mm, corresponding to a "real" structure with dimensions 12 by 16.5 m. The test frames were built of 50 by 60 mm RC-sections. A plane view of the test frames and the test set-up is shown in figure 1. The mass of each frame is $\simeq$ 2 kN. The frames are tested in pairs of two giving three identical set-ups and the last frame is serving as a spare/reference frame. The three set-ups are labelled AAU1, AAU2 and AAU3 respectively. To model the storey deck, 8 RC beams (0.12*0.12*2.0m) are placed on each storey. The total mass per frame is then $\simeq$ 20 kN.

To avoid overlapping longitudinal reinforcement causing uncontrolled variations of bending stiffness and strength anchoring steel-plates welded to the reinforcement are applied to the longitudinal reinforcement bars.

The concrete used has a design compression strength of 20 MPa with a maximum aggregate diameter of 5 mm. For each frame approximately 80 l concrete is used.

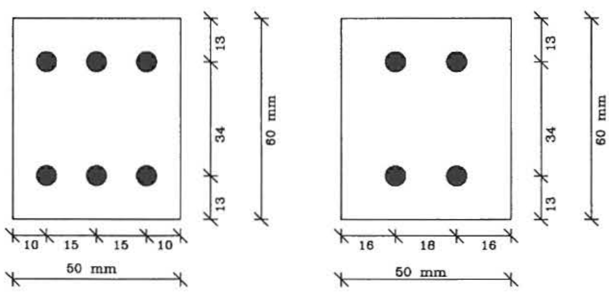

Figure 2: Cross-section of beam and columns.

The dimensions of the beams and columns in the frame are constant all over the frame with outer measures of 50 $\times$ 60 mm.

Columns are reinforced with 6 KS410/D=6mm (ribbed steel) and beams with 4 KS410/D=6mm, see figure 2.

The experiments performed on the frames can generally be divided into two types:

- Non-destructive testing (free decay tests, weak motion excitation)
- Destructive Testing (strong motion excitation)

The non-destructive testing is performed by means of free decay tests and weak motion excitation of the test frames. The free decay tests are performed by applying a horizontal force at the top-storey which is suddenly released and the free decay motions are measured. The weak motion excitation is performed using the time series from the strong motion tests scaled down with a factor of 100.

The destructive testing is performed by applying strong motions to the structure. In the tests three different types of excitations as illustrated in figure 3 are considered.

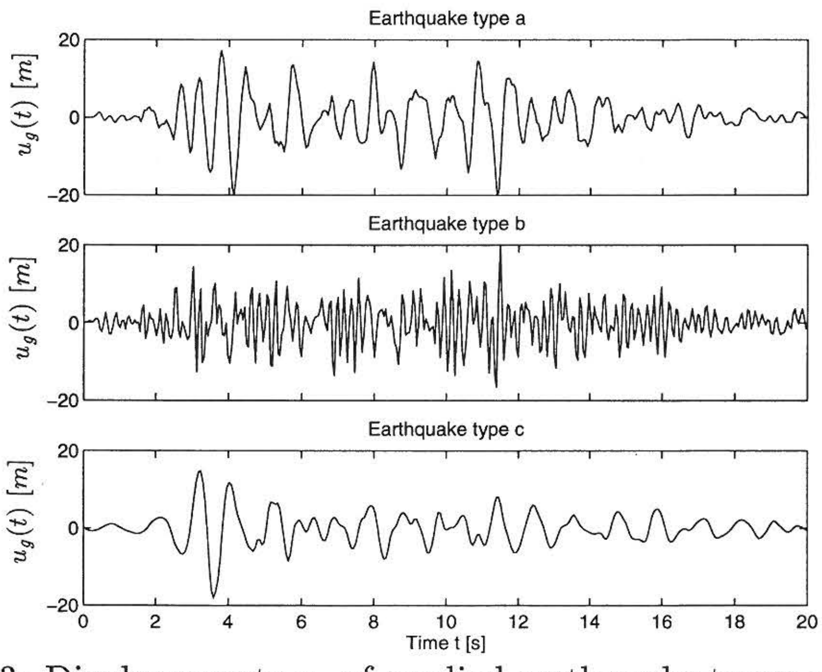

Figure 3: Displacements u_g of applied earthquake types scaled to maximum amplitude of 20 mm.

The type a and b earthquakes are artificially generated using a Kanai-Tajimi filter with centre circular frequencies at ω_a = 10rad/s and ω_b = 30rad/s, respectively. The type c earthquake is a scaled version of an accelerogram measured during the 1994 Northridge earthquake in California. The main difference between the type a,b and the type c earthquake is that the type c earthquake is non-stationary in frequency content. The type c earthquake is only used in the weak motion shaking to test the robustness of the modal identification methods to frequency non-stationary excitations.

Totally three setups, labelled AAU1, AAU2 and AAU3 are tested in the following way: Setup AAU1 was tested until complete failure using earthquake type a in the strong motion tests. Three sequential ground motion series were applied using scaling factors on the series shown in figure 3 of 0.25, 0.5 and 0.75, respectively. During the third earthquake the second and third storey of the structure collapsed. The AAU2 structure was exposed to two sequential earthquakes of type a using scaling factors of 0.2 and 0.4, respectively. Structure AAU3 was exposed to three sequential earthquakes of type b with scaling factors of 0.1, 0.2 and 0.35 respectively.

After the strong motion tests, the structures AAU2 and AAU3 were cut into smaller pieces which were subjected to static tests, see figure 4a. The stiffnessess determined from these static tests were then compared to identical stiffness found from static tests performed on the undamaged reference frame. A schematic view of the static test set-up is shown in figure 4.

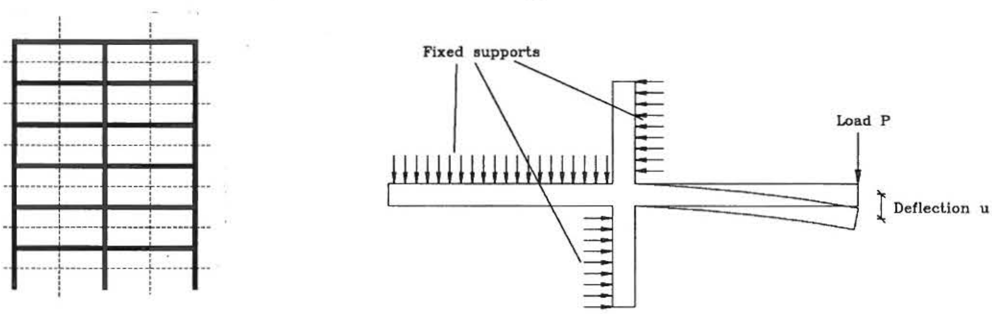

Figure 4: a) Separation lines. b) Schematic set-up of static testing of reference and damaged specimens.

DAMAGE ASSESSMENT

In this paper only the maximum softening/final softening damage indices are considered for the damage assessment, and the theoretical damages are compared to the visually observed damage.

The maximum softening concept is based on the variation of the vibrational periods of a structure during a seismic event. A strong correlation between the damage state of a reinforced concrete structure that has experienced an earthquake and the global maximum softening has been documented. In order to use the maximum softening as a measure of the damage of the structure it is nessecary to establish a quantitative relationship between the numerical value of the maximum softening and engineering features of damage. This relationship is obviously very complicated and has to be found by measurement from real structures by regression analysis. DiPasquale et al. [2] investigated a series of buildings damaged during earthquakes and found a very small variation coefficient for the maximum softening damage index, see figure 5.

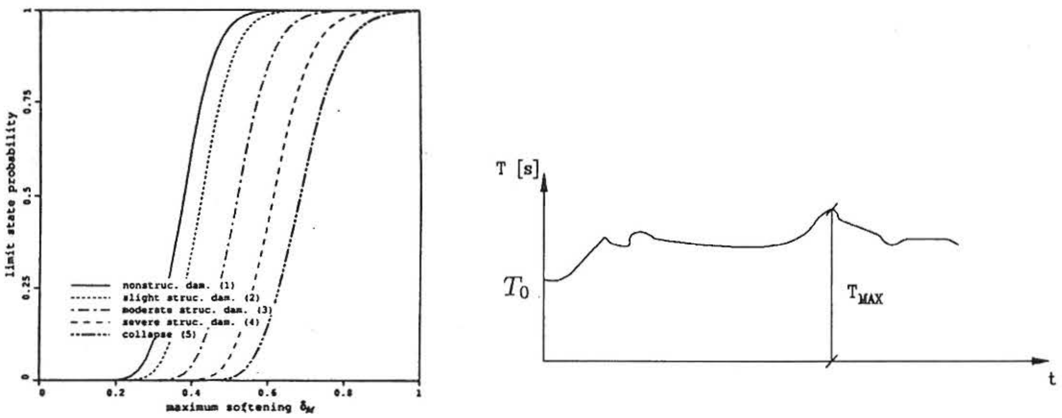

Figure 5: a) Distribution function of observed limit state values of one-dimensional maximum softening reported by DiPasquale et al. [2]. b) Definition of maximum value of the fundamental eigenperiod.

Nielsen and Çakmak [4] extended the maximum softening to substructures based on a multi-dimensional maximum softening $\delta_{M,i}$ defined as

$$\delta_{M,i} = 1 - \frac{T_{0,i}}{T_{M,i}} \tag{1}$$

Where $T_{0,i}$ is the initial value of the ith eigenperiod for the undamaged structure and $T_{M,i}$ is the maximum value of the ith eigenperiod during the earthquake, see figure 5. Explicit expressions for the damage localization were developed for the 2-dimensional case.

It is clear from the definition of this index that in case the maximum softening is 0 no damage has occurred in the structure, and when $\delta_M = 1$ there has been a total loss of global stiffness in the structure.

If the softening damage index given by eq. (1) is evaluated using the final value of the eigenperiods $T_{F,i}$ the index is referred to as the final softening damage index.

After each series of ground motion the entire structure was visually examined and the damage state of each storey of the building was described by marking all cracks and afterwards photos were taken.

EXPERIMENTAL RESULTS

In this section some results obtained from the tests performed on the frame AAU3 are presented.

Initially, the free decay test was performed for identification of the modal parameters of the virgin structure. The modal parameters was extracted using a vector ARMA technique, see Kirkegaard et al. [3]. The identified frequencies are shown in table 1 for the three cases where a pull-out force of 0.25kN, 0.5kN and 0.75kN has been applied. Identical tests with a pull-out force of 0.5kN were performed after each of the earthquakes and these results are listed in table 1 as well.

Case	$f_{1,fd}$ $[Hz]$	$f_{2,fd}$ $[Hz]$	ζ_1 [%]	ζ_2 [%]
Virgin 0.25kN	2.20	7.15	1.7	1.1
Virgin 0.50kN	2.25	7.27	1.5	1.0
Virgin 0.75kN	2.25	7.29	1.4	0.9
After EQ1 0.5kN	1.97	6.39	2.4	1.9
After EQ2 0.5kN	1.73	5.67	3.2	2.4
After EQ3 0.5kN	1.41	4.55	4.6	3.2

Table 1: Evaluated frequencies and damping ratios of the virgin structure and after each earthquake.

During the three earthquakes the top-storey accelerations and displacements were measured as shown in figures 6 and 7, respectively. From the acceleration signals the two lowest time-varying frequencies of the structure were extracted using a recursive implemented vector ARMA model, see Kirkegaard et al. [3]. The evaluated development in the two lowest eigenfrequencies during the three earthquakes are shown in figure 8 and the calculated softenings are shown in table 2.

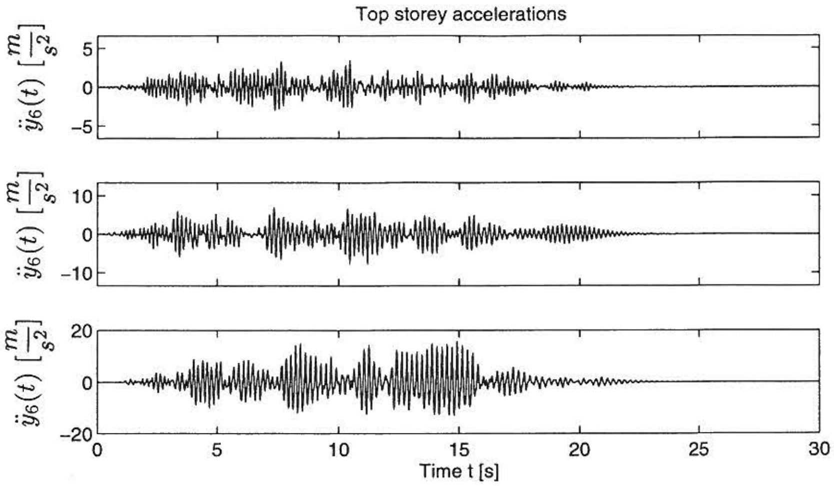

Figure 6: Measured top storey accelerations during the three earthquakes. Setup AAU3.

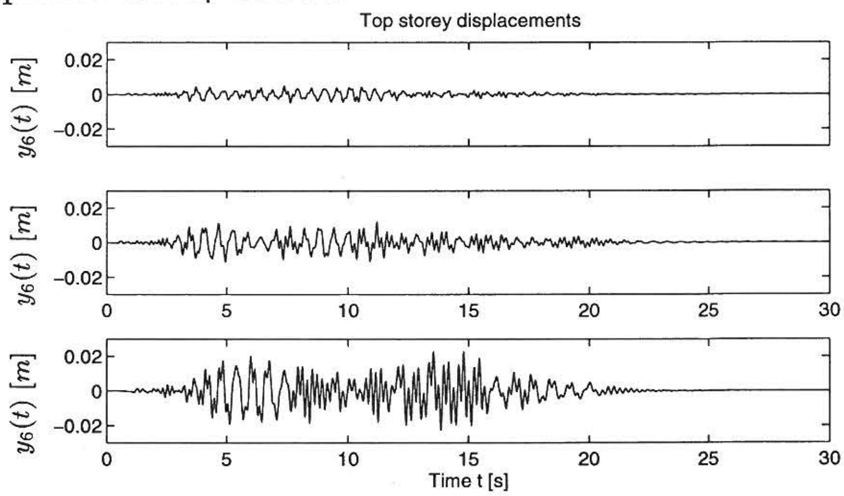

Figure 7: Measured top storey displacements during the three earthquakes. Setup AAU3.

Case	$f_{F,1}$ $[Hz]$	$f_{F,2},$ $[Hz]$	$f_{M,1}$ $[Hz]$	$f_{M,2}$ $[Hz]$	$\delta_{F,1}$	$\delta_{F,2}$	$\delta_{M,1}$	$\delta_{M,2}$
EQ1	1.97	6.39	1.97	6.21	0.12	0.12	0.12	0.15
EQ2	1.73	5.67	1.71	5.64	0.23	0.22	0.24	0.22
EQ3	1.41	4.55	1.41	4.26	0.37	0.37	0.37	0.41

Table 2: Evaluated frequencies and corresponding maximum and final softenings, $\delta_{M,i}$ and $\delta_{F,i}$.

After each of the earthquakes the structure was visually inspected for cracks. After the first earthquake, minor crack development was found at the joints between columns and beams all over the structure with slightly larger cracks at the second, fourth and fifth storey. After the second earthquake the cracks had grown

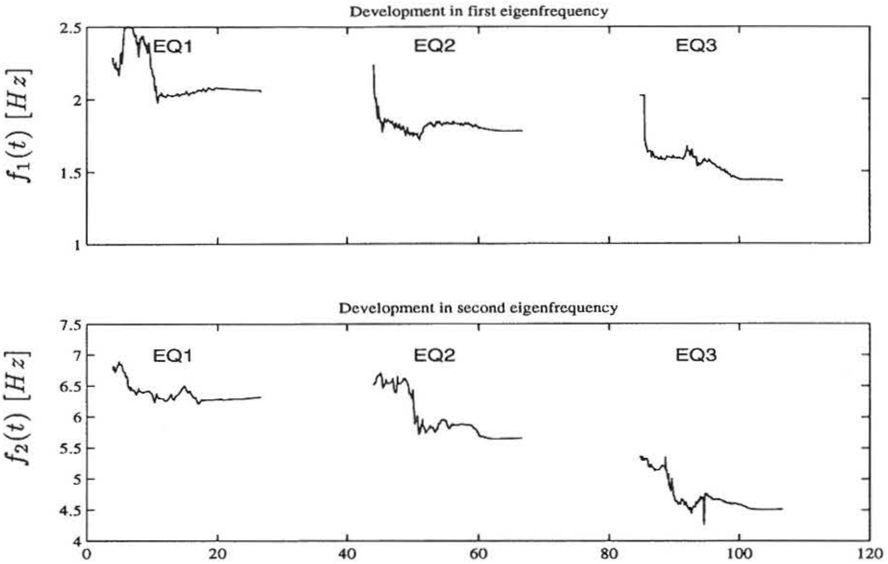

Figure 8: Evaluated development in 1st and 2nd eigenfrequency $f_1(t)$ and $f_2(t)$ during the three earthquakes, set-up AAU3.

significantly, especially at the fourth and fifth storey, where the cracks almost went through the cross-section. After the third earthquake further crack growth was observed along with concrete crushing in the joint between the beams and the middle column in the fourth storey and the damage state could in this case be characterized as severe due to the beginning of concrete crushing. It should here be noted that even though only limited damage was observed visually, the structure can be in a severely damaged state. This was the case for setup AAU1, where only limited cracking and almost no concrete crushing were observed after EQ2, but still the structure failed in the initial parts of EQ3. However, the maximum softening was evaluated as $\delta_{M,1} = 0.39$ during EQ2 indicating that the damage state was critical since $\delta_{M,1}$ was close to the critical value of 0.43 for the maximum softening as indicated in figure 5.

Comparing the evaluated maximum softenings in table 2 and the distribution function shown in figure 5a the damage evaluation based on the maximum softening indicates severe damage in the structure which is in a very good agreement with the visually observed damage.

CONCLUSIONS

In this paper a series of shaking table test was presented. Selected data from one of the structures considered in the test series were presented and processed in order to evaluate modal parameters of the structure and changes in those due to strong motion excitations. The evaluated changes in eigenfrequencies were used to calculate the maximum and final softening damage indicators and these were compared to the visually observed damage. Using the distribution function for the maximum softening suggested by the inventors of this damage indicator revealed very good agreement between the visually observed damage and the values of the maximum softenings.

ACKNOWLEDGEMENT

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