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Earthquake Tests of Reinforced Concrete Frames

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Abstract *The paper presents a series of shaking table experiments performed at the Structural Laboratory at Aalborg University, Denmark during the autumn of 1996. A brief description of the purpose and aim of the tests are given and some selected results are presented. The considered structure is a 2-bay, 6-storey reinforced concrete frame in scale 1:5 with the outer measures 2.4 × 3.3 m. The test structure consists of two identical frames spaced 1000 mm on a shaking table. The dead-load of deck elements and pay load is modelled by 8 RC-beams placed on each storey in span between the two frames. One of the frames is instrumented with accelerometers at each storey and the shaking table is equipped with an accelerometer as well. Initially the modal parameters of the test structure are determined from a free decay test, where the top storey is pulled approximately 5 mm out from the equilibrium state. Afterwards the test structure is subjected to the three strong ground motion oscillations where the two first sequences are followed by a free decay test. No free decay test was performed after the third earthquake due to collapse of the test structure during the third strong motion oscillation. After each of the strong motion sequences the structure is visually examined to reveal cracks etc., and this visual damage assessment is compared to the observed reduction in the two lowest eigenfrequencies of the structure which can be extracted from the measured top storey accelerations during the strong motion events.*

Keywords: Shaking Table Testing, RC-frames, Damage Assessment, Modal Identification.

1 Introduction

When civil engineering structures are subjected to sufficiently high dynamic loads it is well known that some kind of damage will occur in the structure. In RC-structures the damage may start as cracking, eventually developing into crushing of concrete and yielding of reinforcement. The damage may 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 to assess the damage in the structure. 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, 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 this 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 Banon et al. [1] Stubbs et al. [20], Penny et al., [11], Casas [2], DiPasquale et al. [3], Hassiotis et al. [4], Kirkegaard et al. [5], Koh et al. [7], Nielsen et al. [8], Pandey et al. [9], Park et al. [10], Penny et al. [11], Reinhorn et al. [12], Rodriguez-Gomes [13], Skjærbæk et al. [15], [16], Stephens et al. [17], [18], [19] and Vestroni et al. [22].

The motivation for performing earthquake experiments with a scale 1:5 6-storey, 2-bay model test frames are to provide data for verification of methods for non-destructive damage assessment of RC-frames based on measured reponses of the structure during the previous damaging event. The aim of this paper is to show the applicability of the maximum softening damage index as a damage indicator for reinforced concrete frames subject to earthquakes, see e.g. Nielsen et al. [8].

A schematic view of the test set-up is shown in figure 1.

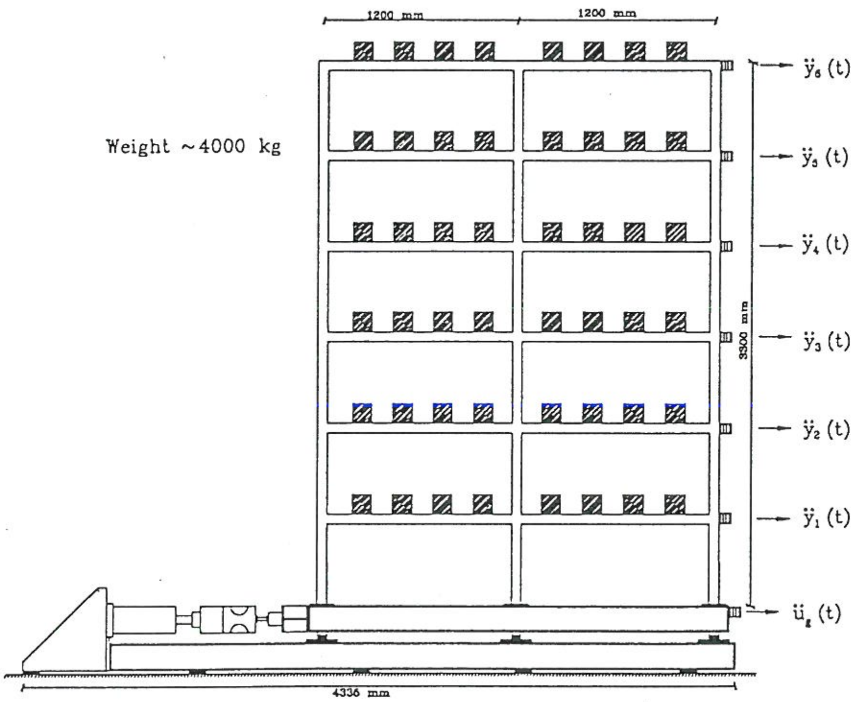

Figure 1: A schematic view of the setup of the considered frame.

The frame considered in this paper is exposed to three earthquake like ground motions of increasing magnitude.

2 Damage Assessment

2.1 The Maximum Softening Damage Index

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 earthquake and the global maximum softening δ_M 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. [3] investigated a series of buildings damaged during earthquakes and found a very small variation coefficient for the maximum softening damage index, see figure 2.

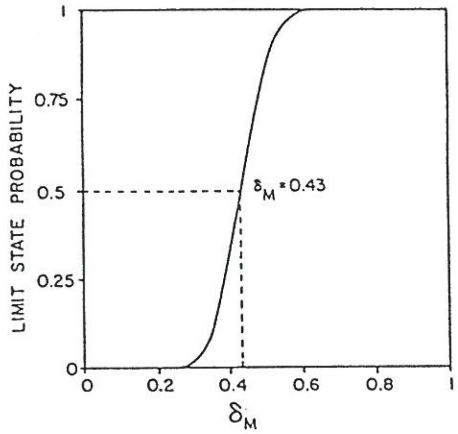

Figure 2: Distribution function of observed limit state values of one-dimensional maximum softening reported by DiPasquale et al. [3].

Nielsen and Çakmak [8] and DiPasquale and Çakmak [3] extended the maximum softening to substructures based on a multi-dimensional maximum softening $\delta_{M,i}$ defined as

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

Where $T_{i,initial}$ is the initial value of the ith eigenperiod for the undamaged structure and $T_{i,\max}$ is the maximum value of the ith eigenperiod during the earthquake, see figure 3. Explicit expressions for the damage localization were developed for the 2-dimensional case.

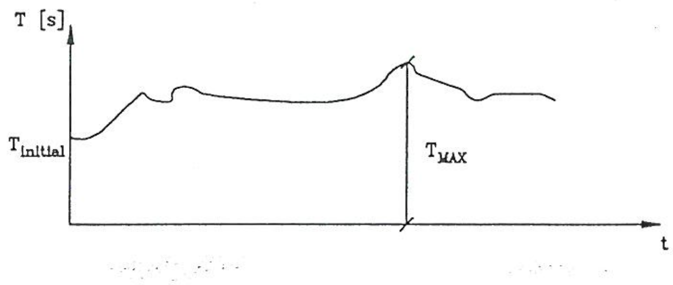

Figure 3: Definition of maximum value of the fundamental eigenperiod.

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.

2.2 Visual Inspection

After each series of ground motions the entire structure is visually examined and the damage state of each storey of the building is given one of the following 6 classifications: Undamaged (U), Cracked (CR), Lightly Damaged (LD), Damaged (D), Severly Damaged (SD) or Collapse (CO). Each of the 6 classifications is defined in table 1.

Category	Definition
Undamaged UD	No external sign of changed integrity of any of the columns or beams in the storey.
Cracked CR	Lightly cracking observed in several members but no permanent deformation.
Lightly Dam. LD	Severe cracking observed with minor permanent deformations.
Damaged D	Severe cracking and local large permanent deformations observed.
Severely Dam. SD	Large permanent deformations observed and spalling of concrete at some members.
Collapse CO	Very large permanent deformations observed and severe spalling of concrete at several members.

Table 1: *Definition of the 6 damage classifications used.*

3 Experimental Results

In this section the obtained measurements are presented and the processed data are shown in order to investigate the correlation between observed damage and the calculated softenings.

3.1 Instrumentation of Frames and Data Acquisition

The considered test frame was equipped with an accelerometer at each storey measuring the horizontal acceleration. Furthermore, the shaking table was equipped with an accelerometer. The data aquisition was performed using an 16 channel HBM data-recorder connected to a PC running the HBM CATMAN program. The sampling was performed at a rate of 150 Hz.

In order to eliminate high frequency noise problems the measured acceleration time series are low-pass filtered using a digital 8th order Butterworth filter implemented in the MATLAB program. The filter was designed with a cut-off frequency of 30 Hz.

3.2 Modal Identification of Virgin Structure

The virgin structure was identified from a free decay test, where a force of approximately 50 kg was applied at the top storey to pull the structure out of the equilibrium state.

The modal parameters of the structure are determined using an AutoRegressive Vector model ARV that is fitted to the 6 measured responses at the six storeys, see Kirkegaard et al. [6].

The estimated frequencies and damping ratios are shown in table 2.

mode no.	f_i [Hz]	ζ_i[%]
1	1.95	2.9
2	6.57	1.7

Table 2: *Estimated modal parameters of the virgin structure.*

It should here be noted that especially the estimates of the second mode seem to be somewhat sensitive to which parts of the free decay are used for the identification. An example of the sampled data from a free decay test is shown in figure 4.

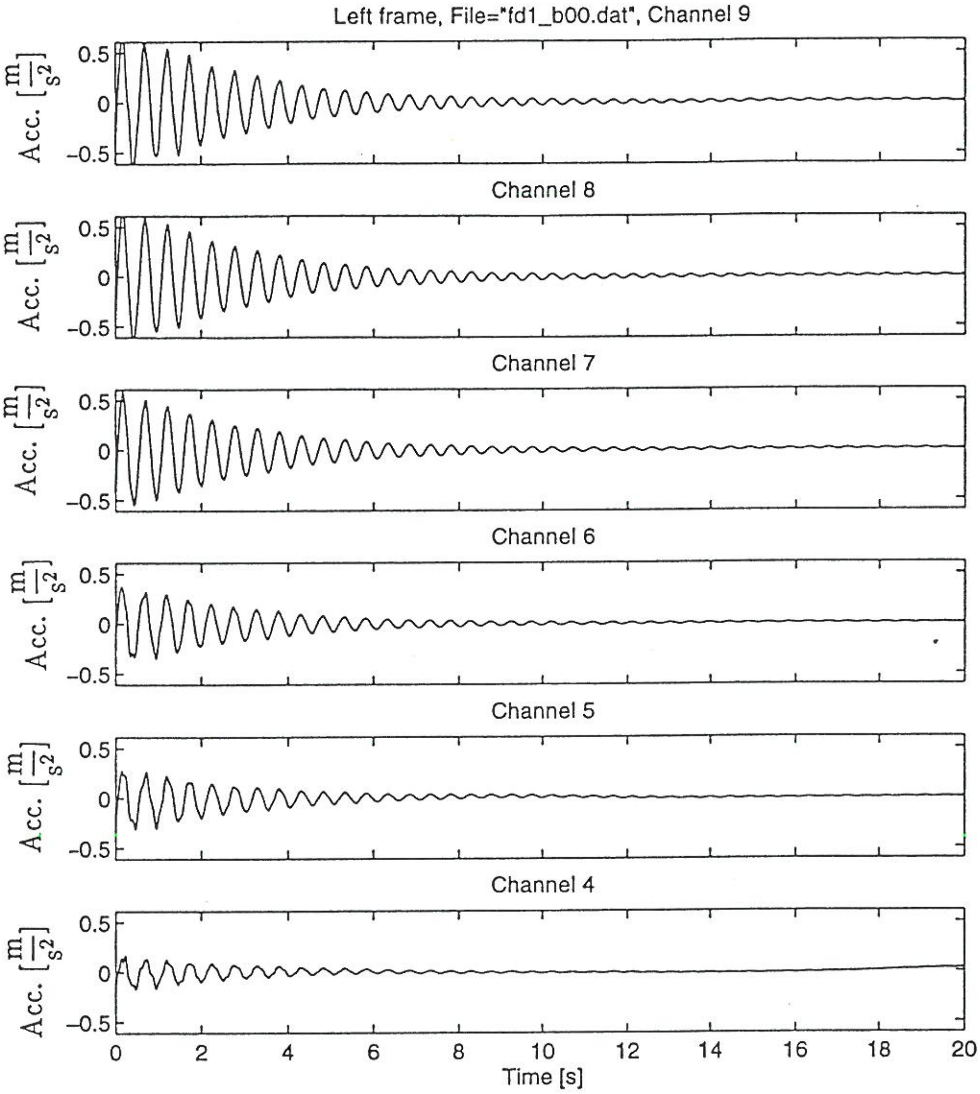

Figure 4: Measured accelerations from pull-out tests of undamaged frame AAU1.

3.3 Strong Motion Tests

During the strong motion part of the experiment the frame is exposed to three sequential earthquake like ground motions of increasing magnitude as shown in figure 5. The earthquakes are modelled using a Kanai-Tajimi filter with a center frequency of 10rad/s and a filter damping of 0.3. The peak-accelerations in the three cases are $-0.14g$, $-0.27g$ and $-0.41g$. After each of the two first earthquakes a free decay test is performed as for the virgin structure.

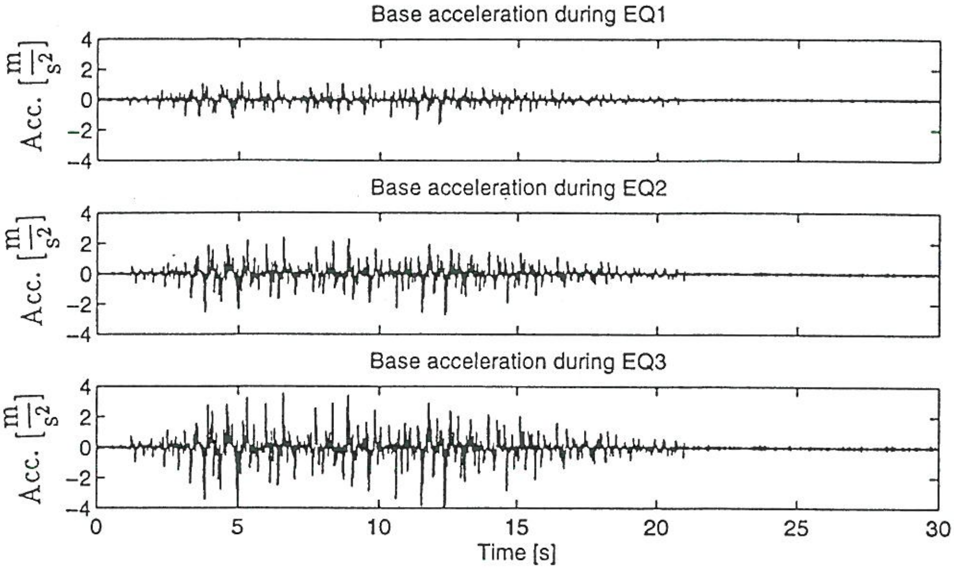

Figure 5: Applied ground motion accelerations during EQ1, EQ2 and EQ3.

During the strong motion events the storey accelerations are measured in the same manner as in the case of the free decay tests. The measured top storey response during the three earthquakes is shown in figure 6. By processing these acceleration time series in an appropriate manner the top storey displacements in figure 7 are obtained, see Skjærbæk et al. [14]. From this figure the decreasing tendency of the fundamental period of the structure is easily seen.

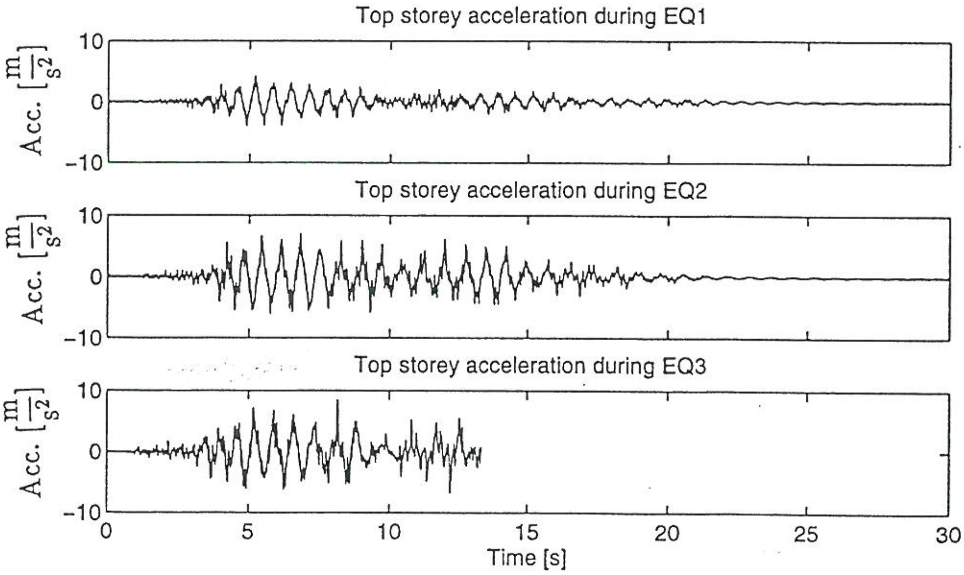

Figure 6: Top storey accelerations during EQ1, EQ2 and EQ3.

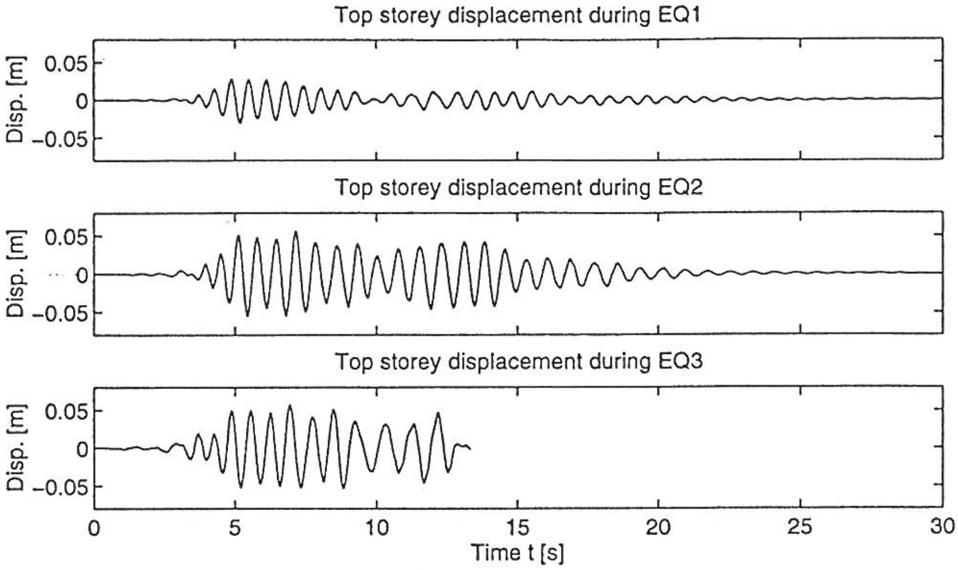

Figure 7: Top storey displacements during EQ1, EQ2 and EQ3.

To identify the modal parameters of the frame as a function of time a recursive implementation of an AutoRegressive MovingAverage Vector model is used. The applied model is a single-input-multi-output model where the estimates are updated sequentially through the time series as new information becomes available. The development in the two lowest frequencies as a function of time is shown in the figures 8-10. It should be noticed that no smoothing of the eigenfrequncies has been performed.

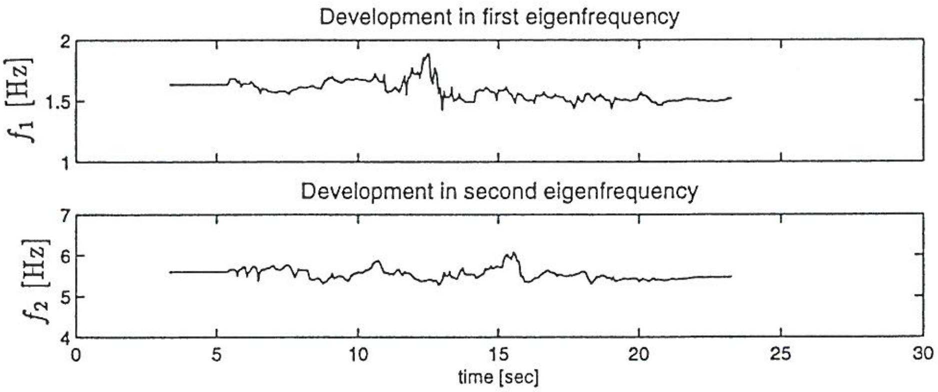

Figure 8: Development of softening in first and second mode during EQ1.

From the figures 8-10 the maximum softenings in the first and second mode can be found and the results are shown in table 3.

	$f_{\min,1}$ $[Hz]$	$f_{\min,2}$ $[Hz]$	$\delta_{M,1}$	$\delta_{M,2}$
EQ1	1.42	5.28	0.27	0.20
EQ2	1.19	4.71	0.38	0.28
EQ3	0.96	4.37	0.51	0.33

Table 3: *Estimated minimum frequencies and maximum softenings during the three earthquakes.*

The results from the free decay tests performed after EQ1 and EQ2 are shown in table 4.

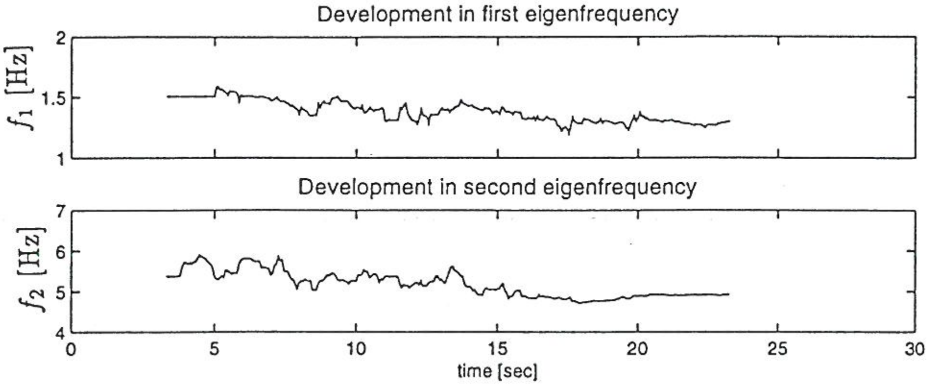

Figure 9: Development of softening in first and second mode during EQ2.

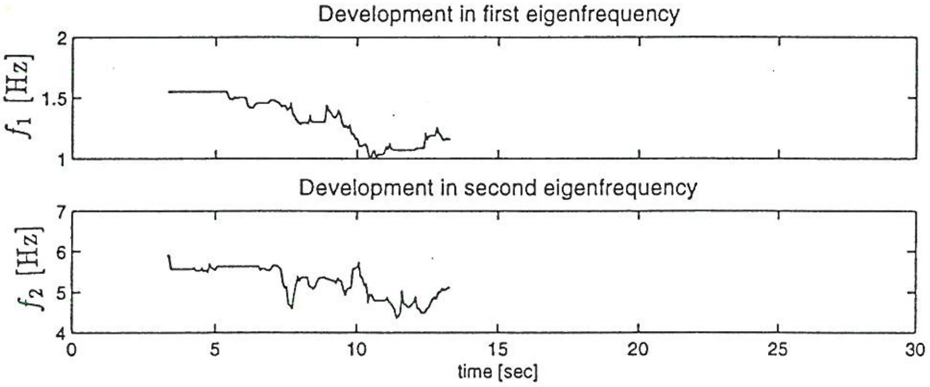

Figure 10: Development of softening in first and second mode during EQ3.

From the estimated minimum frequencies in table 3 and the frequencies estimated from free decay tests (table 4) after each of the earthquake events it is seen that during the earthquake the structural eigenfrequencies are lower than the structural frequencies found for the damaged structure after the earthquake. This indicates that yielding in some of the structural members occur during the earthquake. From table 2 it is seen that the maximum softening in the first mode right before the collapse of the structure is approximately 0.5. This value of the maximum softening is in good agreement with the damage state of a severe damaged structure, see figure 2.

earthquake No.	f_1 [Hz]	f_2 [Hz]	ζ_1[%]	ζ_2[%]
EQ1	1.58	5.62	3.8	2.2
EQ2	1.32	5.01	4.5	2.2

Table 4: *Estimated modal parameters of the damaged structure after EQ1 and EQ2.*

3.4 Visual Inspection of Test Structure

After each of the strong motion events the structure was visually inspected by means of a magnification glass where all cracks were marked by a pencil.

Generally the cracks/damage were concentrated at the beam-column connections at all load levels and the inspection was therefore concentrated at the nodes. The general impression from the visual inspection after both the first and second strong motion oscillations was that the damage was limited since only small cracks were present.

Pictures of the cracks at the middle node at the second storey and at the right node at the second storey are shown in figures 11 and 12 after EQ1 and in figures 13 and 14 after EQ2. Only at the nodes in the second and third storeys significant localized permanent deformations were found. During the third strong motion event both the second and third storey collapsed completely and the first storey suffered severe damage. A picture of the collapsed structure is shown in figure 15. A classification of the damage as described in section 2.2 is given in table 5.

Storey	EQ1	EQ2	EQ3
1	UD	CR	SD
2	CR	LD	CO
3	CR	LD	CO
4	UD	CR	LD
5	UD	CR	CR
6	UD	CR	CR

Table 5: *Damage classifications after the three earthquake events.*

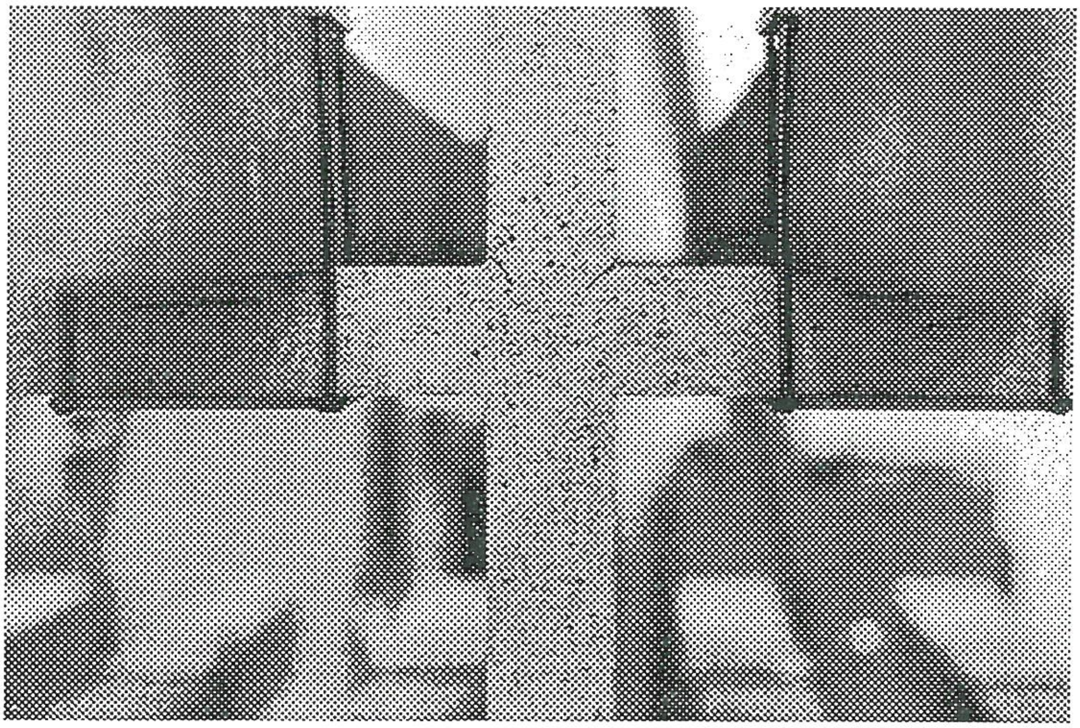

Figure 11: Cracks in node 5 after EQ1.

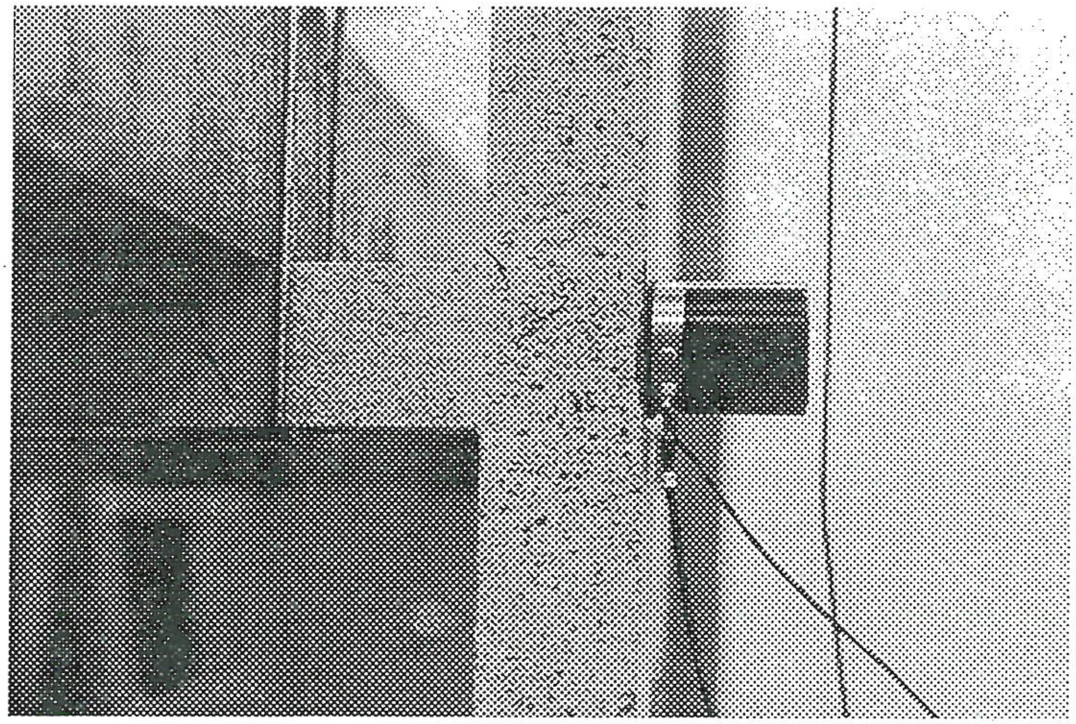

Figure 12: Cracks in node 6 after EQ1.

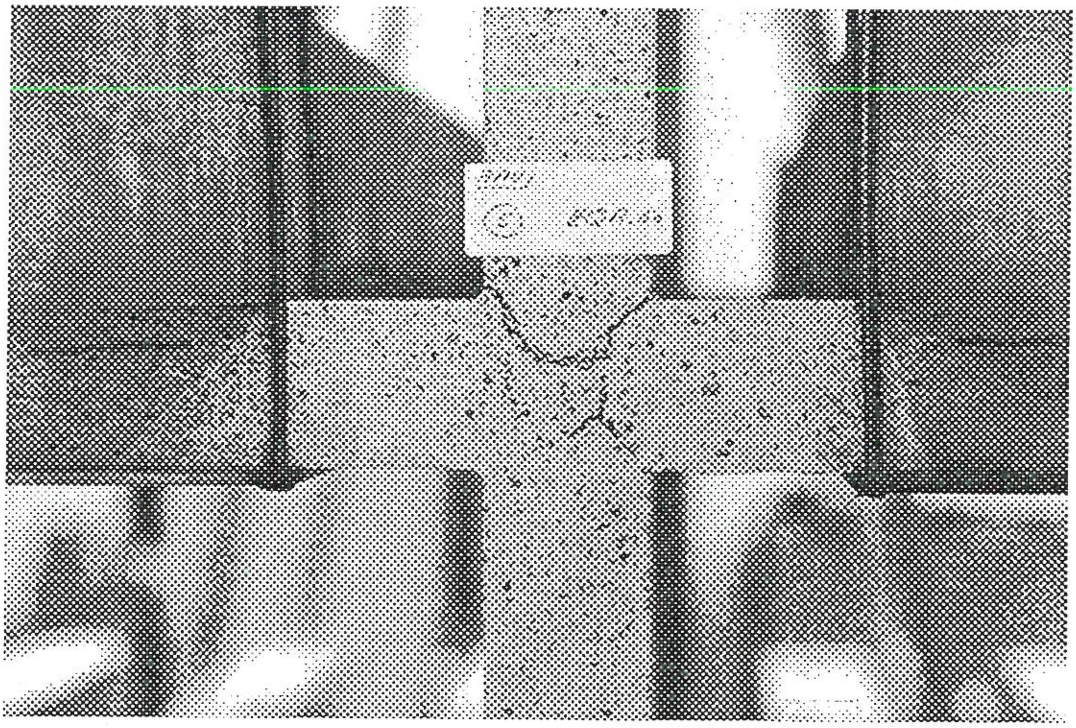

Figure 13: Cracks in node 5 after EQ2.

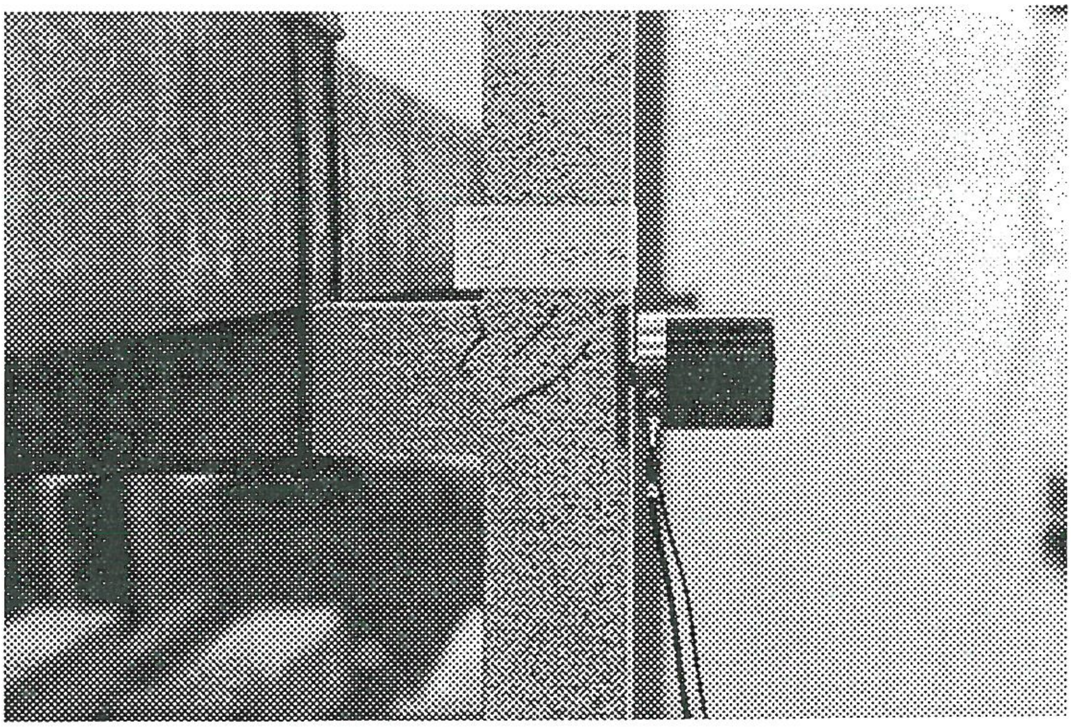

Figure 14: Cracks in node 6 after EQ2.

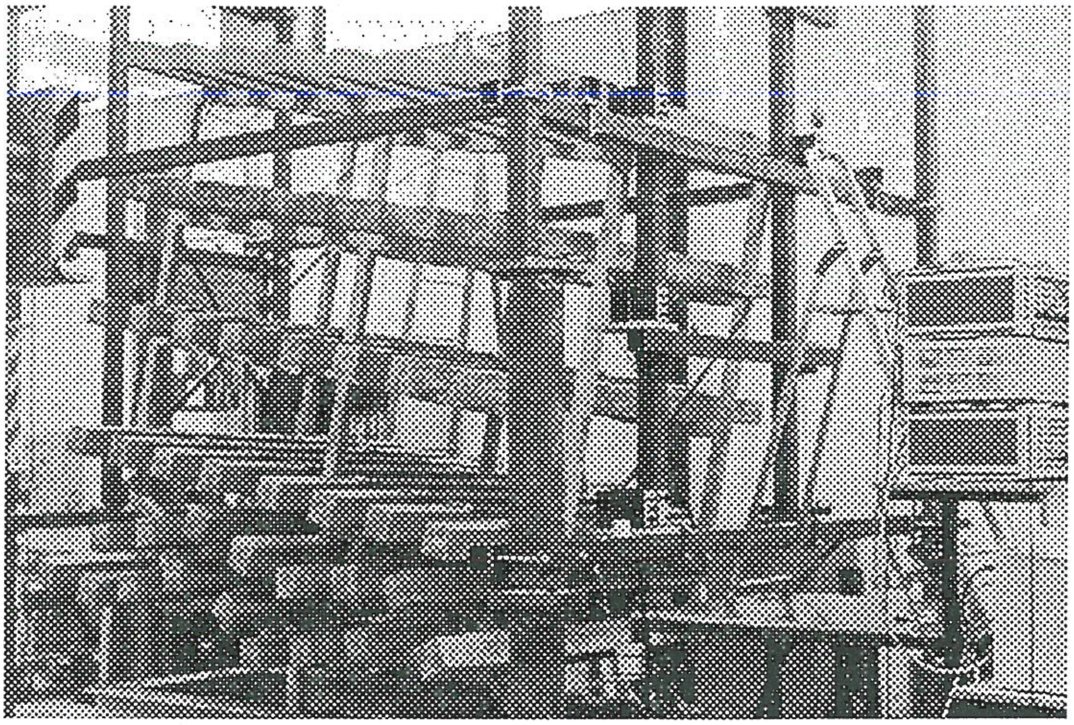

Figure 15: Collapsed structure after EQ3.

By comparing the results of the visual inspection with the evaluated maximum softenings in table 3 the magnitude of the maximum softenings clearly indicate larger structural stiffness changes than the visual inspection. After EQ1 the results of the visual inspection were that only a few small cracks had been initiated and that the structure was basically undamaged. However a maximum softening of 0.28 in the first mode indicates an average stiffness change of approximately 50 per cent. This can be explained by the fact that the damage introduced in the structure during this earthquake is mainly of internal character, such as slippage between concrete and reinforcement.

4 Conclusions

In the paper a series of shaking table tests with a scale 1:5 reinforced concrete frame is considered. Visual damage assessment of the structure was compared to damage assessment based on the two-dimensional maximum softening damage index, and it was found that the considered frame structure showed significant values of the maximum softening even though basically no external damage was observed during the visual inspection. Higher values of the maximum softening in the first mode than in the second mode was found and this indication of the highest damage level in the lower part of the structure is supported by the visual inspection where the largest cracks were found in the three lowest storeys.

5 Acknowledgement

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