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An Experimental Study of a Midbroken 2-Bay, 6-Storey Reinforced Concrete Frame subject to Earthquakes

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Abstract A 2-bay, 6-storey model test RC-frame (scale 1:5) subjected to sequential earthquakes of increasing magnitude is considered in this paper. The frame is designed with a weak storey, in which the columns are weakened by using thinner and weaker reinforcement bars. The aim of the work within this paper is to study the global response to a damaging strong motion earthquake event of such buildings. Special emphasis is put on examining to what extent damage in the weak storey can be identified from global response measurements during an earthquake where the structure survives and what level of excitation that is necessary in order to make the weak storey identification. Furthermore, emphasis is put on examining how and where damage develops in the structure and especially how the weak storey accumulates damage. Besides the damage in each storey, the structure is identified by a static load at the top storey while measuring the horizontal displacement of the stories and also visual inspection is performed.

Key words: Model Testing, Reinforced Concrete Structures, Damage Assessment, Modal Identification, Midbroken Structures.

1 Introduction

Dynamic loads seen in the nature such as earthquakes, wind, flood, and etc. cause damages to civil engineering structures. Since there is no escape from this reality, the only thing that can
be done for an existing structure is to control the damage growth by suitable localization and quantification procedures. In most earthquake codes, the acceptable level of damage is specified depending on the strength of the earthquake, but the main idea is to prevent total collapse of a structure or total collapse of a part of a structure and make it repairable, so the demolition of the structure can be avoided. This point of view led to a new research area known as "damage assessment" resulting in many different methods in the literature in the recent 10-20 years.

In reinforced concrete, (RC), structures, damage under dynamic loadings usually starts as cracks followed by crushing of concrete or yielding of reinforcement, in case the structure does not suffer any other kind of construction failure. Traditionally, assessment of damage in RC-structures is made 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. But developments in earthquake engineering and especially developments in the recent 10-20 years of damage assessment procedures, offer a much more attractive method which basically depends on measuring the structural response at given locations of the structure. Almost all of the methods developed according to this idea are based on calculating a so-called damage index, which is supposed to reflect the damage state of the considered structure, substructure or structural member by the use of e.g. changes in dynamic characteristics. 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. [26], Penny et al., [18], Casas [2], DiPasquale et al. [3], Hassiotis et al. [5], Koh et al. [11], Pandey et al. [14], Park et al. [16], [17], Penny et al. [18], Reinhorn et al. [19], Rodriguez-Gomes [20], Skjærbaek et al., [21], [22], Stephens et al. [24], [25] and Vestroni et al. [27].

Recent earthquake events have revealed a class of structures which can be referred to as mid-broken structures. (Pan Cake type of damage). As a definition a mid-broken structure has a storey weakened by some kind of change in the material used or changes of the geometry of members’ cross-sections. Examples of this phenomenon are found in Turkey and Greece, where buildings are often built in several stages. After the construction of each stage, reinforcement bars are left extended through the next storey and finally through the concrete deck at the roof. When the construction work is resumed, perhaps years later, these extended reinforcement bars are used to connect and anchorage of the new structural components to the existing building. Recent earthquake events in Turkey and Greece have shown that such structures have a tendency to fail in the storeys where such a connection is performed. The same kind of failure mode was also seen in the Kobe 1995 earthquake, where the mid-storey collapses were caused by a sudden change of stiffness and strength of the storey columns. An example of such a failure can be seen in Figure 1 below.
Figure 1: Collapse of an intermedia floor due to the October 1995 Earthquake, Dinar, Turkey.

Until now only very limited research has been dedicated to this phenomenon, and the studies have been limited to numerical simulation studies by e.g. Skjærbæk et al. [23], and Kölüoğlu et al. [9], [10]. The results of these simulations indicate that the increased amount of damage in a weak storey, is mainly due to the strength degradation. However, some divergence has been found between different models used in the simulations and experimental verification of the simulated results are desired.

The aim of this paper is to present an experimental investigation on the effect of a weak storey in a structure. Special emphasis is put on investigating to what extent the weak storey can be identified from global response measurements during an earthquake where the structure survives and what level of excitation is necessary to make the weak storey identification.

2 The Test Structure

For the test series 2 reinforced concrete frames were cast one at a time and were constructed identically. The test frame considered is a 6-storey, 2-bay RC-frame. The dimensions of the test frame is 2460 by 3300 mm corresponding to a ”real” structure with dimensions 12.3 by 16.5 m. The test frame is built of 50 by 60 mm RC-sections constant all over the frame. The weight of each frame is \( \approx 2 \) kN. To model the storey deck, 8 RC beams (0.12*0.12*2.0m) are placed on each storey. The total weight per frame is then \( \approx 20 \) kN which makes about 40 kN totally for the test structure.

The longitudinal reinforcement used in the frame are of the type KS550 (ribbed steel) with an average yield stress of 610 MPa. In the weak fourth storey St37 steelbars were used with an average yield strength of 390 MPa. Columns and beams are reinforced with 4Φ6 KS550 and in the weak storey, columns are reinforced with 4ø5.5 St37. A plane view of the test frame and the reinforcement of the cross-sections can be seen in figures 2a and 2b, respectively.
The Test Structure

Columns

Weight ~ 4000 kg

Figure 2: Plane view of experimental set-up and cross-sections of beams and columns.

In figures 3a and 3b typical stress-strain curves are shown for the two types of steel.

Figure 3: Stress-strain curves for the used types of reinforcement.

To avoid overlapping in the longitudinal reinforcement giving uncontrolled changes in bending stiffness and strength, the ends of the longitudinal reinforcement bars are provided with anchoring steel-plates welded to the reinforcement.

The concrete used has a design compression strength of 30 MPa with a maximum aggregate diameter of 5 mm. The modulus of elasticity and the ultimate stress determined from the test cylinders’ compression tests is approximately found as $f_c = 45$ MPa and $E = ??$ MPa.

All columns and beams are reinforced against shear with 2 mm steel thread which has been formed into spirals as seen in figure 4.
3 Test Set-up and Conduction of Dynamic Tests

The frames are tested in pairs of two where the same ground surface acceleration is applied to the two frames. The frames are placed at the shaking table at a distance of 1000 mm and are stabilized in space by a steel cross at each end. In the connection between the shaking table and the hydraulic cylinder a load cell is placed to measure the actual cylinder force as a function of time. Furthermore, the cylinder is capable of measuring the cylinder displacements as a function of time. Both the right and the left hand frames were equipped with an accelerometer measuring the horizontal acceleration at each of the storeys. Also the shaking table is equipped with an accelerometer measuring the horizontal ground accelerations. Furthermore, a load-cell and a displacement transducer are attached to the hydraulic cylinder measuring the applied force and the ground displacements, figure 2a.

3.1 Non-Destructive Testing

The non-destructive testing is performed by means of free decay tests in which a horizontal force at the top storey of the frame is applied and where the structure is identified from well defined data. Free decay tests are performed in the virgin state of the structure and subsequent to each of the earthquake events.

3.2 Destructive Testing

The destructive testing is performed by applying three sequences of earthquake like ground motions to the model test structure. The acceleration versus time diagrams are given in figure 5.
The time series shown in figure 5 were generated by filtering white noise through a Kanai-Tajimi filter with a circular frequency of $30 \text{ rad/s}$ and a damping ratio of 0.1. This centre frequency is likely to excite primarily the first and second mode of the structure.

4 Non-Destructive Dynamic Testing

In this section the results of the non-destructive testing of the frame in the virgin state and the results after each strong motion event are presented. Before strong motions are applied the frame is subjected to various loads in the linear range to provide data for modal identification of the original structure. Furthermore, free decay tests are performed after each earthquake to provide "clean" data for identification of the damaged structure. The frame is subjected to free decays of pull-outs in bending where a load of 0.25 kN, 0.50 kN, 0.75 kN has been applied step by step.

The free decay test time series were analyzed using an AutoRegressive Vector model (ARV), Eigen Realization Algorithm (ERA), the Polyreference method (POLYREF) and an AutoRegressive Moving Average Vector (ARMAV) model, see Pandit [15], Juang [6], Vold [28] and Kirkegaard [8].
The modal parameters listed in tables 1, 2, 3 and 4 were obtained with respect to the virgin state and after EQ1, EQ2 and EQ3, respectively. In table 1, the results are listed for all four methods in the case of a pull-out force of 0.25 kN and it can be seen that in case of frequencies all methods practically give the same results. The estimation of the damping ratios seem to be somewhat poorer. The remaining estimates of the modal parameters are listed as averages of the results obtained by the four methods.

<table>
<thead>
<tr>
<th>Method</th>
<th>State</th>
<th>( f_1 ) [Hz]</th>
<th>( f_2 ) [Hz]</th>
<th>( \zeta_1 ) [%]</th>
<th>( \zeta_2 ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ARV</td>
<td>0.25 kN</td>
<td>2.68</td>
<td>8.66</td>
<td>1.12</td>
<td>0.54</td>
</tr>
<tr>
<td>ERA</td>
<td>0.25 kN</td>
<td>2.68</td>
<td>8.66</td>
<td>1.14</td>
<td>0.35</td>
</tr>
<tr>
<td>POLYREF</td>
<td>0.25 kN</td>
<td>2.67</td>
<td>8.66</td>
<td>1.19</td>
<td>0.70</td>
</tr>
<tr>
<td>ARMAV</td>
<td>0.25 kN</td>
<td>2.68</td>
<td>8.66</td>
<td>1.24</td>
<td>0.51</td>
</tr>
<tr>
<td>Average</td>
<td>0.50 kN</td>
<td>2.64</td>
<td>8.58</td>
<td>1.35</td>
<td>0.82</td>
</tr>
<tr>
<td>Average</td>
<td>0.75 kN</td>
<td>2.62</td>
<td>8.51</td>
<td>1.48</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Table 1: Estimated modal parameters for virgin frame AAU4.

<table>
<thead>
<tr>
<th>Method</th>
<th>State</th>
<th>( f_1 ) [Hz]</th>
<th>( f_2 ) [Hz]</th>
<th>( \zeta_1 ) [%]</th>
<th>( \zeta_2 ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>0.25 kN</td>
<td>2.31</td>
<td>7.46</td>
<td>2.15</td>
<td>1.44</td>
</tr>
<tr>
<td>Average</td>
<td>0.50 kN</td>
<td>2.22</td>
<td>7.18</td>
<td>2.92</td>
<td>1.90</td>
</tr>
<tr>
<td>Average</td>
<td>0.75 kN</td>
<td>2.16</td>
<td>7.01</td>
<td>3.22</td>
<td>1.94</td>
</tr>
</tbody>
</table>

Table 2: Estimated modal parameters of frame AAU4 after EQ1.

<table>
<thead>
<tr>
<th>Method</th>
<th>State</th>
<th>( f_1 ) [Hz]</th>
<th>( f_2 ) [Hz]</th>
<th>( \zeta_1 ) [%]</th>
<th>( \zeta_2 ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>0.25 kN</td>
<td>1.89</td>
<td>5.91</td>
<td>3.13</td>
<td>2.62</td>
</tr>
<tr>
<td>Average</td>
<td>0.50 kN</td>
<td>1.78</td>
<td>5.59</td>
<td>3.85</td>
<td>3.63</td>
</tr>
<tr>
<td>Average</td>
<td>0.75 kN</td>
<td>1.72</td>
<td>5.43</td>
<td>4.59</td>
<td>4.48</td>
</tr>
</tbody>
</table>

Table 3: Estimated modal parameters of frame AAU4 after EQ2.
From tables 1 and 2 it can be seen that the frequency drops by approximately 14% after EQ1 while the damping ratio increases by approximately 100%. These values are 35% and 400%, respectively after EQ2 and 45% and 350%, respectively after EQ3, which is also obvious from figure 6.

<table>
<thead>
<tr>
<th>Method</th>
<th>State</th>
<th>$f_1$ [Hz]</th>
<th>$f_2$ [Hz]</th>
<th>$\zeta_1$ [%]</th>
<th>$\zeta_2$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>0.25 kN</td>
<td>1.55</td>
<td>4.83</td>
<td>3.52</td>
<td>2.27</td>
</tr>
<tr>
<td>Average</td>
<td>0.50 kN</td>
<td>1.42</td>
<td>4.50</td>
<td>5.11</td>
<td>3.06</td>
</tr>
<tr>
<td>Average</td>
<td>0.75 kN</td>
<td>1.38</td>
<td>4.36</td>
<td>4.52</td>
<td>3.30</td>
</tr>
</tbody>
</table>

Table 4: Estimated modal parameters of frame AAU4 after EQ3.

As an example the top storey acceleration versus time diagrams are plotted after each of the four types of loadings and can be seen in figure 6.

Figure 6: The top storey acceleration versus time diagrams from the free decay tests. a) Undamaged structure, b) After EQ1, c) After EQ2 and d) after EQ3.
Using the the four presented methods it is possible to obtain estimates of the mode shapes as well. The results of the mode shape identification of the virgin structure and of the damaged structure after each earthquake are shown in figure 7 for a pull-out force of 0.5 kN using the ARV method.

Figure 7: Mode shapes of the structure. [---]: Virgin state, [---]: After EQ1, [---]: After EQ2 and [---]: After EQ3.

No significant changes in the mode shapes were obtained after any of the earthquake events as indicated in figure 7. Although there is a weak storey in the structure the mode shapes seem to be insensitive to even large stiffness changes.
5 Destructive Dynamic Testing

In this section data collected during and after the strong motions applied to the structure are presented. In the destructive testing the frame AAU4 is exposed to three sequential earthquakes of increasing amplitude called EQ1, EQ2 and EQ3. In figure 8, measured accelerations at the top storey during the three earthquakes can be seen, respectively.

![Figure 8: Top storey accelerations during the three earthquakes.](image)

5.1 Processed Data

This section presents processed data where inter-storey and top-storey displacements have been found using double time integration procedures as described in e.g. Skjærbæk [23]. During the integration process where displacements are obtained a Butterworth 6th order high-pass digital filter with a cut-off frequency of 0.95 Hz and a Butterworth 8th order low-pass digital filter with a cut-off frequency of 20 Hz have been used. Furthermore, time series of eigenfrequencies are extracted from the strong motion records. The procedure for frequency estimation is described in Kirkegaard et al. [7]. In figure 9 the top storey displacements during EQ1, EQ2 and EQ3 are shown.
Figure 9: Top storey displacements relative to ground surface during EQ1, EQ2 and EQ3.

The displacement time series shown in figure 9 indicates that the global structural response during EQ1 is very small since the top storey displacement is smaller than 10 mm. As the structure becomes more damaged and the amplitude of the earthquake increases it is seen that the global structural response is increasing quite dramatically.
In the same manner as for the top storey displacements, the interstorey drifts are evaluated and these are shown in figures 10, 11 and 12 for the three earthquakes, respectively.

The interstorey displacements shown in figure 10 indicate that the main response is within the second, third and fourth storey where the largest amplitudes are found. Especially, the response of the sixth storey is very limited and any damage observed in this storey is likely to be due to cracking of hitherto uncracked sections.
During EQ2 the interstorey displacements increase significantly in all storeys and only the third storey seems to have significantly smaller amplitudes than the rest of the storeys. The maximum interstorey drift is found to be 10 mm in storey 5.
In the third earthquake the interstorey displacements increase further but the same distribution as during EQ2 is found between the storeys. So, based on the observed interstorey displacements the weak storey cannot be found since no significant increase in interstorey displacements is observed.
In order to evaluate the development of the natural frequencies of the structure during the earthquakes a recursive implemented ARMAV model has been fitted to the measured acceleration time series, see Kirkegaard et al. [7]. In figure 13 the development in the two lowest eigenfrequencies of the structure are shown.

![Figure 13: Development of eigenfrequencies in the first and second mode during EQ1, EQ2 and EQ3.](image)

For each of the eigenfrequency time series the maximum softening damage index is evaluated. This index has proven to be a good measure of the global damage state of a reinforced concrete structure, see DiPasquale et al. [4], Nielsen et al. [12], [13].

Generally the multi-dimensional maximum softening $\delta_{M,i}$ is defined according to Nielsen et al. [12] as

$$\delta_{M,i} = 1 - \frac{T_{i,\text{initial}}}{T_{i,\text{max}}}$$  \hspace{1cm} (1)$$

Where $T_{i,\text{initial}}$ is the initial value of the $i$th eigenperiod for the undamaged structure and $T_{i,\text{max}}$ is the maximum value of the $i$th eigenperiod during the earthquake.

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 14. The maximum softenings during the three runs are shown in table 5.
Figure 14: Distribution function of observed limit state values of one-dimensional maximum softening reported by DiPasquale et al. [3].

<table>
<thead>
<tr>
<th></th>
<th>$f_{\text{min},1} [\text{Hz}]$</th>
<th>$f_{\text{min},2} [\text{Hz}]$</th>
<th>$\delta_{M,1}$</th>
<th>$\delta_{M,2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ1</td>
<td>2.07</td>
<td>6.54</td>
<td>0.22</td>
<td>0.24</td>
</tr>
<tr>
<td>EQ2</td>
<td>1.57</td>
<td>5.15</td>
<td>0.41</td>
<td>0.40</td>
</tr>
<tr>
<td>EQ3</td>
<td>1.31</td>
<td>4.49</td>
<td>0.50</td>
<td>0.48</td>
</tr>
</tbody>
</table>

Table 5: Estimated minimum frequencies and maximum softenings during the three earthquakes.

6 Static Tests

In order to evaluate the distribution of the damage, the structure is exposed to static tests after each earthquake event. The tests were performed with the entire structure. In the static tests, a definite force is applied at the top storey and the displacements are measured at each storeys. The force is varied between 0 to 0.75 kN. A schematic view of the test set-up is shown in figure 15.

Figure 15: Static test set-up used for determination of lateral stiffness.
The static testing was performed on the entire structure in the virgin state, EQ1, EQ2 and EQ3. So the results of all the static tests are shown in figure 16.

Figure 16: Force-deformation curves for interstorey displacements obtained from the static tests of the frame. [—]: Virgin structure, [— — —]: After EQ1, [— . — . —]: After EQ2 and [— .. —]: After EQ3.

As seen from figure 16, the interstorey displacement of the fourth storey is about 2.5 times higher than the average of the rest of the storeys in the virgin state, 2.7 times higher after EQ1 and 2.5 times higher after EQ2. As expected, the deformations increase a lot after EQ3. It should be noted that there is also a soft behaviour at the top storey after EQ3 but still the relative displacement of the fourth storey is about 2.2 times higher than the average displacement value of the rest of the storeys. In table 6, the stiffnesses found for each of the storeys are shown for the virgin structure as well as the structure after EQ1, EQ2 and EQ3. Based on the stiffnesses determined in table 6, a damage indicator \( \delta_{S,i} \) for the \( i \)th storey can be represented as

\[
\delta_{S,i} = 1 - \sqrt{\frac{K_{\text{final},i}}{K_{0,i}}} \tag{2}
\]
Table 6: Evaluated stiffnesses of the 6 storeys.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Virgin [N/mm]</th>
<th>EQ1 [N/mm]</th>
<th>EQ2 [N/mm]</th>
<th>EQ3 [N/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1499</td>
<td>1390</td>
<td>1010</td>
<td>932</td>
</tr>
<tr>
<td>2</td>
<td>2124</td>
<td>1456</td>
<td>1054</td>
<td>622</td>
</tr>
<tr>
<td>3</td>
<td>1870</td>
<td>1326</td>
<td>991</td>
<td>622</td>
</tr>
<tr>
<td>4</td>
<td>778</td>
<td>554</td>
<td>431</td>
<td>334</td>
</tr>
<tr>
<td>5</td>
<td>1486</td>
<td>1486</td>
<td>895</td>
<td>592</td>
</tr>
<tr>
<td>6</td>
<td>3339</td>
<td>1544</td>
<td>1160</td>
<td>548</td>
</tr>
</tbody>
</table>

where $K_{0,i}$ is the apparent stiffness of the $i$th storey of the undamaged structure and $K_{final,i}$ is the apparent stiffness of the $i$th storey after the forcing event. It should be noted that this formulation of the damage index, $\delta_{S,i}$, is compatible with the maximum softening index according to Nielsen et al. [12], [13].

The damage indicators calculated after each earthquake event according to this criterion are listed in table 7.

Table 7: Calculated damage indices after each earthquake.

<table>
<thead>
<tr>
<th>Storey</th>
<th>$\delta_{S,i}$ (EQ1)</th>
<th>$\delta_{S,i}$ (EQ2)</th>
<th>$\delta_{S,i}$ (EQ3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.04</td>
<td>0.18</td>
<td>0.21</td>
</tr>
<tr>
<td>2</td>
<td>0.17</td>
<td>0.30</td>
<td>0.46</td>
</tr>
<tr>
<td>3</td>
<td>0.16</td>
<td>0.27</td>
<td>0.42</td>
</tr>
<tr>
<td>4</td>
<td>0.16</td>
<td>0.26</td>
<td>0.34</td>
</tr>
<tr>
<td>5</td>
<td>0.00</td>
<td>0.22</td>
<td>0.37</td>
</tr>
<tr>
<td>6</td>
<td>0.32</td>
<td>0.41</td>
<td>0.59</td>
</tr>
</tbody>
</table>

The damage indicator calculated for the sixth storey after EQ1 shows a very high damage level compared to the other storeys. This can be explained by the fact that initial stiffness obtained from the pull-out tests of the storey is very high which indicates that the elements in this storey are still uncracked although the other storeys are cracked. This interpretation is also supported by studying the observed interstorey drifts during EQ1 in the sixth storey, where only very small deformations are found. Based on this assumption new damage indicators for the sixth storey are found to $\delta_{EQ1}^{S,6} = 0.00$, $\delta_{EQ2}^{S,6} = 0.13$ and $\delta_{EQ3}^{S,6} = 0.40$.

In figure 17, the total deformations for each storey at the maximum load level are shown.
Figure 17: Total deformations at maximum load. [——]: Virgin structure, [— — —]: After EQ1, [— – – –]: After EQ2 and [⋯]: After EQ3.
7 Visual Inspection of Test Structure

After each of the strong motion loads, the structure is thoroughly examined visually by means of a magnification glass where all cracks were marked with different colours.

7.1 Definition of the Classifications

After each series of ground motions the structure is thoroughly visually examined and the damage state of each storey of the building is classified into one of the following 6 classifications: Undamaged (U), Cracked (CR), Lightly Damaged (LD), Damaged (D), Severely Damaged (SD) or Collapse (CO). Each of the 6 classifications are defined in table 8.

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

Table 8: Definition of the 6 damage classifications used.

7.2 Damage Assessment of Frames AAUWa-b

Generally the cracks/damage were concentrated at the beam-column junctions at all load levels and the inspection was therefore concentrated at the nodes. After earthquake 1, (EQ1), the inspection made on the nodes showed no serious cracks other than mostly micro-cracks. Only the nodes 2, 12 and 18 of frame AAUWb had shear cracks not more than 0.02 mm wide. These cracks increased in length after EQ2 and also new shear cracks occurred at the nodes 8 and 18 of frame AAUWa and 1, 8 and 15 of frame AAUWb. So the general impression from these visual inspections after EQ1 and EQ2 was that the damage was limited since only small cracks and some reasonable shear-cracks were present.

EQ3 resulted in some heavy damages at the nodes of the second and the fourth storeys which can be seen from the pictures in figure 18. Other than these heavy damages, most of the nodes had significant shear cracks. All three columns of the fourth storey of frame AAUWb had horizontal shear cuts where the maximum shear force is supposed to be. Storey 2 also had the same kind of damage.
There was no total or partial collapse after any of the earthquake events. But the nodes 2, 11 and 14 of the frame AAUWa and the nodes 2, 11 and 14 of AAUWb can be presented as hinges. An overview of these visual inspections is given in table 9 below.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Frame</th>
<th>EQ1</th>
<th>EQ2</th>
<th>EQ3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>AAUWa/AAUWb</td>
<td>UD/CR</td>
<td>CR/LD</td>
<td>SD/SD</td>
</tr>
<tr>
<td>2</td>
<td>AAUWa/AAUWb</td>
<td>UD/CR</td>
<td>CR/CR</td>
<td>LD/D</td>
</tr>
<tr>
<td>3</td>
<td>AAUWa/AAUWb</td>
<td>CR/UD</td>
<td>LD/CR</td>
<td>LD/D</td>
</tr>
<tr>
<td>4</td>
<td>AAUWa/AAUWb</td>
<td>CR/CR</td>
<td>CR/CR</td>
<td>SD/SD</td>
</tr>
<tr>
<td>5</td>
<td>AAUWa/AAUWb</td>
<td>UD/UD</td>
<td>CR/CR</td>
<td>SD/SD</td>
</tr>
<tr>
<td>6</td>
<td>AAUWa/AAUWb</td>
<td>UD/CR</td>
<td>CR/CR</td>
<td>CR/LD</td>
</tr>
</tbody>
</table>

Table 9: Damage classifications after the three earthquake events for frame AAUW.

It is quite clear from table 9 that the first, fourth and the fifth storeys are the most damaged ones, while storey six has the least damage. Especially the shear cuts seen in all the three columns of the fourth storey, prove that the storey has weaker columns than the others and the columns are almost in the ultimate limit state of their load bearing capacities.
Figure 18: Photos of all nodes in frame AAUWb.
8 Summary

The results of a series of shaking table tests with a scale of 1:5, 6 storey reinforced concrete frame designed with a weak fourth storey have been presented. Initially the structure was exposed to various non-damaging tests in order to identify both interstorey stiffnesses and modal parameters of the virgin structure. Afterwards three sequential earthquakes of increasing amplitude were applied to the structure. After each of the earthquake events the structure was again exposed to non-damaging tests to evaluate changes in interstorey stiffness and modal parameters. From the strong motion measurements the two lowest time varying eigenfrequencies of the structure were estimated as function of time and the maximum softening was evaluated for the two modes. After the first earthquake (EQ1) sequence the maximum softening was evaluated to 0.22 and 0.24 in the first and second mode, respectively. According to the fragility curves for the maximum softening damage index presented in the literature this level for the maximum softening corresponds to only "non-structural damage". The visual inspection of the structure after the first earthquake revealed only development of a few micro cracks in all storeys other than the fifth. The static testing of the storeys revealed stiffness changes in the four lower storeys and the sixth storey, where the stiffness change in the sixth storey can be referred to internal cracking since the response measurements only showed a very low level of interstorey displacements in this storey. The mode shapes identified after the first earthquake were found only to change slightly, whereas the structural damping ratio in both the first and the second mode increased approximately 100%.

After the second earthquake (EQ2) a maximum softening of 0.40 and 0.41 were found in the first and the second mode, respectively. According to the fragility curves this corresponds to a "light to moderate damage" state of the structure. The visual inspection revealed cracking spread out in the entire structure and light damage was observed in the beam-columns junctions above the first and the third storeys. The static tests showed stiffness changes in all storeys with the highest level in the second, third and fourth storey. Again the identified modeshapes only revealed small changes but damping ratios were increased approximately 300% compared to the virgin structure. After the third earthquake (EQ3) a maximum softening of 0.50 and 0.48 were found in the first and second mode, respectively, and this level corresponds to "severe damage" in the structure. The visual inspection revealed severe damage with crushing of concrete in the beam-column junctions above the first, fourth and fifth storey whereas the static tests showed the largest reduction in stiffness in the second, third and sixth storey. But in none of the applied methods for damage assessment the weak fourth storey was found to be significantly more damaged than any of the other storeys, even though both stiffness and strength of this storey were significantly lower than the other storeys in the virgin structure. No significant change was found in the mode shapes from EQ2 to EQ3 and the damping ratios remained at a level approximately 3-4 times higher than the virgin structure.

Closer values of the maximum softenings $\delta_{M,i}$ in the first mode and in the second mode of the structure indicates that all the storeys have damage and this is also supported by the visual inspections. Visual inspections, maximum softenings and the static damage indicator $\delta_{s,i}$ are compatible with each other after EQ1. After EQ2 and EQ3 there were differences between visual inspections and the $\delta_{s,i}$ damage indicators. The reason of these differences are that...
when local damage such as hinges occur the loss of stiffness becomes more clear at a certain distance from the actual location of the damage. E.g. when a hinge develops in the fourth storey the apparent stiffness of the fifth storey may decrease significantly which can express more damage and increase the displacements in the fifth storey.

9 Conclusions

In this paper the results a series of shaking table tests with a model test 6 storey frame structure designed with a weak fourth storey have been presented. The weak storey was obtained by reducing the amount of reinforcement and using a poorer steel quality. In the undamaged structure the horizontal stiffness of the weak storey was found to be approximately 50% of the stiffness in the remaining storeys, in means of less strength and stiffness.

The general conclusion from this study is that a weak storey in terms of changes in reinforcement did not make any significant effects on the dynamic displacements of the structure since no significant increases were seen in the interstorey displacement of the weak storey. Furthermore, the mode shapes also did not show any signs of reduction in stiffness in the fourth storey. From visual inspection of the structure after the last earthquake, the fourth storey was found to be severely damaged e.g. the mid columns in both frames were having hinge-like joints to the third and the fifth storeys and shear-cuts where the maximum shear force is supposed to be.

The damage evaluation from static tests showed a very low stiffness in the fourth storey after the last earthquake, but the relative change in stiffness were almost identical in all storeys.

From the test performed in this study, it can be concluded that the structures having weak mid-storeys have tendency of developing severe damage in these storeys and this also effects the behavior and the magnitude of the displacements especially in the storeys above the weak storey where severe damage was found as well.

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