Experimental study of earthquake sequence effect on structures

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ABSTRACT: It is common that structures experience sequences of earthquakes. A sequence can consist of a main shock followed by a series of aftershocks which may have a magnitude high enough to induce damage in a structure. Another situation is the history of earthquakes that a structure may experience throughout its lifetime. This study focuses on the effect of a sequence of earthquakes on a structure, especially on the cumulative damage. For this purpose, SDOF scaled models were tested using a shake table. Two specimens made of RC and two specimens of coir fibre RC were considered. Two different sequences of real seismic records are investigated: the Chilean earthquakes representing a series of major earthquakes a structure may experience throughout its lifetime and the Kobe main shock followed by three aftershocks to simulate the effect of aftershocks. Damage assessment is undertaken using as indicators the change in the fundamental frequency obtained from an impact test and the energy dissipated throughout each earthquake sequence by evaluating the accumulated area of hysteresis cycles.

1 INTRODUCTION
Structures in seismically active zones are unlikely to experience just one earthquake throughout their lifetime, but instead will experience a sequence of earthquakes. This could refer to the multiple earthquakes of different magnitudes that a structure will experience or to a sequence consisting of a main-shock and aftershocks with intensity high enough to induce additional damage in a structure.

Research has been undertaken on the effect of sequences of earthquakes on structures. However, the study limited mainly to numerical analysis (Hatzigeorgiou and Beskos, 2009). In one of the few experimental studies (Moaveni et al., 2006) a full scale building was progressively damaged using four historical earthquake records. The damage was identified by changes in natural frequencies. It was found that the frequency decreased with increasing damage levels. This is expected as damage can be related to a decrease in stiffness.

Another proven indicator of damage progression is hysteretic energy. Cosenza et al. (2009) investigated the relation between the hysteretic energy dissipated during a reversible cyclic excitation series with the cumulative damage observed in a structure. Using this approach cumulative hysteretic energy will be calculated to assess damage progression.

The aim of our study is to analyze the effect that sequences of earthquakes have on a structure. The damage will be assessed by two parameters: change in frequency and hysteresis energy dissipation. It is expected that through measuring the behaviour of a structure during a sequence of earthquakes, appropriate design considerations could be developed and adopted to design, retrofit or repair of a building to resist both the main shock and subsequent aftershocks.

2 SPECIMEN DESIGN

2.1 Scaling procedure
In this work a scaled model of a single-degree-of-freedom (SDOF) system was developed utilizing dimensional analysis. Model scaling is often carried out in engineering research. This is because full scale model testing can be expensive and impractical. Scaled models with equivalent properties
provide a more suitable and cost-effective solution. Through the application of Buckingham’s $\pi$ theory using traditional dimensional analysis, fundamental relationships between the critical physical parameters that define modal responses were formulated (Krawinkler and Moncarz, 1982). Scale modelling requires that the replica model simulates all aspects of the prototype, but this is usually very hard to achieve.

The idealised prototype corresponds to a single story, single bay RC building with a roof of 3.75 m x 3.75 m. The four supporting columns were made of reinforced concrete which had a cross-section of 250 mm x 250 mm and a compressive strength ($f'_c$) of 40 MPa. The height of the building was 3 m and the concrete roof was 100 mm thick. The seismic weight of the structure was determined by the load combination of G+0.3Q according to NZS 1170:2005.

The physical parameters of this prototype were mass (m), height (h), Young’s Modulus (E), acceleration (a) and frequency (f). The basic parameters to be considered in the dimensional analysis and subsequent scaling procedure were length, mass and time. The dimensionless groups generated were:

$$\pi_1 = \frac{E}{m a} \quad \pi_2 = \frac{a f^2}{h}$$

For the experiment an equivalent scaled SDOF model was derived that has the same dynamic properties as the reinforced concrete prototype, and has the same dimensionless parameters. Since the dynamic characteristics were the most important for the earthquake response, an accurate scaling of frequency, mass, acceleration and time was preferred over a consistent scaling of geometric properties. Instead, the geometric properties were changed in accordance with the frequency required. The scale factors used is given in Table 1.

<table>
<thead>
<tr>
<th>Original Structure and SDOF Model</th>
<th>Scale Factor</th>
<th>Scaled Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>m (kg)</td>
<td>3500</td>
<td>0.034</td>
</tr>
<tr>
<td>h (mm)</td>
<td>3000</td>
<td>0.200</td>
</tr>
<tr>
<td>E (MPa)</td>
<td>27898</td>
<td>0.842</td>
</tr>
<tr>
<td>a (g)</td>
<td>1.00</td>
<td>0.996</td>
</tr>
<tr>
<td>f (Hz)</td>
<td>10.8</td>
<td>2.232</td>
</tr>
</tbody>
</table>

2.2 Physical model

Four specimens were tested in this study. Two of them built in plain reinforced concrete (PC) and the other two built in coir fibre reinforced concrete (CFRC). The difference in the behaviour of PC and CFRC are outside of the scope of this article and they are covered by the study of Ali et al. (2011).

The idealized scale models for both fibre and plain concrete had a cross-section of 100 mm x 100 mm and a height of 600 mm as displayed in Figure 1. A lumped mass of 120 kg was situated at the top of the column.

The small scale of the specimens required the use of 8 mm basalt aggregate and PAP 7 to prevent spalling. The mix design adopted for plain concrete was aimed to achieve a compressive strength of 25 MPa (McDermott, 2010). The same mix design was used for CFRC with the addition of 5 cm long coir fibres. Coir fibres were added with a content of 1% of total mass of mixture. The fibre preparation, casting of CFRC and pouring of CRFC into moulds was performed as described by Ali and Chouw (2009).

For the model design the critical factor that had to be considered was the capacity of the shake table. This required the reinforcement ratio to be keep to a minimum whilst still allowing a ductile failure through compliance with minimum reinforcement regulations. This required the use of 6 mm grade 300E longitudinal rebar and 4 mm grade 300E stirrups. The adopted concrete cover was 8 mm.
2.3 Material properties

To determine the actual material properties six cylinders and three small beams were cast with each batch of concrete (PC and CFRC). The basic static properties for both types of concrete were determined by standard procedures in accordance with NZS 3112 (New Zealand Standards, 1986). Cylinders (100 mm diameter x 200 mm height) were tested to determine compressive strength (f’c), and modulus of elasticity (E). Another set of cylinders were tested to determine the splitting tensile strength (STS). Also, a set of small beams (100 mm x 100 mm x 500 mm) were tested to obtain the modulus of rupture (MOR). Table 2 shows the resulting static properties.

The differences between the CFRC and the PC specimens may be due to the presence of a high concentration of fibres and the addition of water to create a workable mixture.

<table>
<thead>
<tr>
<th></th>
<th>E (GPa)</th>
<th>f’c (MPa)</th>
<th>MOR (MPa)</th>
<th>STS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>40</td>
<td>38</td>
<td>4.87</td>
<td>3.49</td>
</tr>
<tr>
<td>CFRC</td>
<td>24</td>
<td>20</td>
<td>3.49</td>
<td>2.46</td>
</tr>
</tbody>
</table>

Figure 2 displays the reinforcement cage in the mould.

3 SHAKE TABLE TESTING

3.1 Test set-up

The set up for the shake table testing is shown in Figure 3. Figure 4 shows the location of the accelerometers and the displacement laser beam used to record the specimen’s response during the test. The accelerometers were used to record the response time history at the top of the specimen and the actual input base acceleration of the shake table. The displacement laser beam was used as back up to verify the response recorded at the top of the specimen.
3.2 Seismic input

Two ground motion sequences were considered. The first consisted of the 1985 Chile earthquake main shock followed by an aftershock of the same seismic event and then the 2010 Chile earthquake main shock (Figure 5). All of these were recorded at the same station (Viña del Mar) and in the same direction. This sequence was to represent the earthquakes which a structure could possibly experience over its lifetime. Although, it may be arguable that repairs would be conducted after a large magnitude earthquake and the structure should be restored to their original non-damaged condition, it is important to note that there may be non-visible damage that would not be repaired or the repairing might be deficient. Therefore an analysis of this case study is still pertinent. The Kobe sequence (Figure 6) consisted of the main shock followed by the three most significant aftershocks. This sequence was used to study the effect of aftershocks. All the records came from the same station (JMA951) in the same direction. Both the Chile and Kobe sequences of records were chosen as they had a frequency content that was likely to affect the specimen and associated large aftershocks so that significant damage would be likely.
The two main shocks in the Chile sequence and the main shock of the Kobe sequence were scaled to have the same spectral acceleration for the specimen’s fundamental frequency (considered as an average of the CFRC and PC specimen). The aftershocks of both the sequences were scaled in the same manner such that their spectral acceleration would be 40% of the scaled main-shocks amplitude. Because of shake table inconsistencies not all aftershocks satisfied this requirement.

Additionally, the records had time scale factors applied to them using the time scale factor that as given in Table 1. The magnitudes of the acceleration were not altered to match the prototype-to-model scaling criteria as the corresponding scale factor presented in Table 1 approximates equal to 1.0.

### 3.3 Base excitation

The base excitation was applied in an incremental manner. The scaled records were applied to the specimens in increments of 10% of displacement until collapse. As the CFRC specimens had a much lower strength this resulted in an earlier failure of the specimen. Modal testing using a calibrated hammer was undertaken between each earthquake loading to identify the fundamental frequency at the corresponding test stage. The Kobe sequence was applied 13 and 8 times for the PC and CFRC, respectively. The Chilean sequence applied 13 and 3 times for the PC and CFRC, respectively. Since a large number of sequences is realistic as past earthquakes have shown, e.g. Darfield earthquake, it was decided to continue the increments until collapse to gain a good indication of damage trends.

### 4 EXPERIMENTAL RESULTS

#### 4.1 Results analysis

The damage will be assessed firstly through modal parameter monitoring and trend identification, specifically: fundamental frequency changes. The cumulative dissipated energy will be used as a measure of structural accumulated damage due to the repeated load cycles.

The frequency trends for the four specimens were obtained experimentally from the calibrated hammer test. For each specimen three hammer impacts were applied at mid height between each earthquake. To achieve accurate results from the accelerometer readings of the three impacts each impact was analysed individually and the average of the three was used. The results were analysed in terms of the fundamental frequency normalized by the specimen’s fundamental frequency at undamaged state.

The acceleration response time-histories at the top of the column, transformed into displacement through a Fast Fourier Transform procedure, were used to find the total displacement while the recorded acceleration at the base was used to find the actual applied displacement. Subtracting the applied displacement from the total displacement gave the relative displacement that was used to evaluate the dissipated energy. The force used to calculate the dissipated energy was based on the assumption that force equals mass times acceleration. This used a mass equal to the mass at the top plus half the column weight and used the acceleration time history from the accelerometer at the top of the column. The energy dissipated during each earthquake or individual ground motion was calculated. As absolute energy values have little meaning, the cumulative dissipated energy throughout each test was normalized on a scale from zero to one, with zero being no dissipated energy and one being total dissipated energy.

Figures 7 to 10 present both the dissipated energy and normalised frequency trends for both PC and CFRC under both the Chilean and Kobe sequences.

#### 4.2 Results discussion

Frequency decay is evident during all four tests. This frequency decrement results from a decrease in stiffness of the structure which reflects the damage. It is particularly clear during the Kobe sequence (Figures 7 and 8) that displays significant and regular drops in frequency associated to the effect of the main shock. In the case of Chile sequence (Figures 9 and 10), the frequency presents a relatively constant rate of degradation consistent with the fact that the Chile sequence consisting of two strong...
main shocks and only one smaller magnitude aftershock.

Even though, only slight decrements in frequency can be seen due to aftershocks during the majority of the tests, this proves that aftershocks, although in magnitude smaller than main shocks, can still cause significant (or at least measurable) damage to a structure.

The slight increments in the fundamental frequency observed in some cases can be explained by random measurement errors and changes in environmental conditions such as ambient noise. It may also be caused by aggregates settling and better aggregate interlock produced by the action of the aftershocks which results in a higher stiffness and consequently a higher frequency.

Similarly to the frequency trend, large increases in dissipated energy can be seen after main shocks in the two tests where the Kobe sequence was applied (Figures 7 and 8). In these tests the contribution of the aftershocks to the accumulated hysteresis energy appears to be negligible. This phenomenon is manifested in flat sections of the energy curves.

Figure 9 does not have large increments in dissipated energy but rather appears to have a relatively constant trend. The “stepped” pattern is primarily due to the aftershock in the middle of the two main shocks having negligible effects on the amount of dissipated energy. Similar trends in damage can be seen from both the energy dissipation and the frequency decrement. However, aftershocks appear to have more effect on frequency than dissipated energy.

Figure 7: Analysis results for the Kobe sequence applied to a CFRC specimen

Figure 8: Analysis results for the Kobe sequence applied to a plain RC specimen

Figure 9: Analysis results for the Chilean sequence applied to a plain RC specimen.
The relationship between the dissipated energy and frequency was also investigated. All the tests exhibited similar logarithmic trends as can be seen in Figure 11. The steep gradient on the left hand side of the graph is in the earlier stages of damage. This part of the graph shows a large frequency decrement with only a very small amount of energy dissipated. On the right hand of the graph, shows a low gradient throughout the four tests, which means that significant amounts of energy are dissipated with only a small change in frequency. This condition occurs when severe damage is observed. It can be seen that the amount of energy increases with only a small change in frequency.

It can be concluded that frequency degradation may be a better indicator at the earliest stage of damage which may consist of non-visible degradation, while energy dissipation may be a better indicator at advanced stages of damage.

5 CONCLUSIONS
This study examined the effect of sequences of earthquakes on structures. Both coir fibre reinforced concrete (CFRC) and plain reinforced concrete (PC) were tested using two different sequences of ground motions. The effect of the sequences is evaluated through damage assessment using the change in frequency and dissipated energy as damage parameters.

Examination of the results leads to the conclusion that frequency decrement and energy increments did occur with damage. These increments/decrements are emphasized when the main shocks are applied.
A decrease in frequency was also evident due to aftershocks, but the effects of aftershocks on dissipated energy appeared negligible.

The comparison of the two damage parameters shows that frequency is more sensitive to earlier stages of damage while cumulative dissipated energy is more sensitive to more severe stages of damage. This is consistent with the previous conclusion that frequency is more sensitive to the effect of aftershocks on structures because this kind of excitations is more likely to generate less significant damage.

Further studies into the effect of earthquake sequences on structures could include the development of a successful numerical model to evaluate the effect of particular seismic records on structures by focusing on the consequence of aftershocks using a larger number of aftershocks as it usually occur in the reality after the destructive earthquake.

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REFERENCES:


