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Considerations on the seismic performance of pre-1970s RC buildings in the Christchurch CBD during the 4th Sept 2010 Canterbury earthquake: was that really a big one?

S. Pampanin, W.Y. Kam, A.S. Tasligedik, P. Quintana Gallo, U. Akguzel

University of Canterbury, Christchurch, New Zealand.

ABSTRACT: The 4th of September 2010 M_w 7.1 Darfield (Canterbury) earthquake had generated significant ground shaking within the Christchurch Central Business District (CBD). Despite the apparently significant shaking, the observed structural damage for pre-1970s reinforced concrete (RC) buildings was indeed limited and lower than what was expected for such typology of buildings. This paper explores analytically and qualitatively the different aspects of the 'apparent' good seismic performance of the pre-1970s RC buildings in the Christchurch CBD, following the earthquake reconnaissance survey by the authors. Damage and building parameters survey result, based on a previously established inventory of building stock of these non-ductile RC buildings, is briefly reported. From an inventory of 75 buildings, one building was selected as a numerical case-study to correlate the observed damage with the non-linear analyses. The result shows that the pre-1970s RC frame buildings performed as expected given the intensity of the ground motion shaking during the Canterbury earthquake. Given the brittle nature of this type of structure, it was demonstrated that more significant structural damage and higher probability of collapse could occur when the buildings were subjected to alternative input signals with different frequency content and duration characteristics and still compatible to the seismicity hazard for Christchurch CBD.

At the time of review/writing of this paper, the 22nd February 2011 M_w 6.2 Christchurch earthquake occurred 10km from the Christchurch CBD. A very brief preliminary observation is included herein as it is very relevant to the discussion of this paper.

1 INTRODUCTION

The M_w 7.1 Darfield (Canterbury) earthquake on the 4th of September 2010 had generated significant ground shaking in the Christchurch Central Business District (CBD), some 35km away from the Darfield epicentre. Peak ground accelerations (PGA) up to 0.19-0.34g were measured in the four recording stations within the CBD area (Cousins and McVerry 2010). The associated seismic demand was deemed to correspond to the approximate 400-500 years return period motions of the New Zealand Loading Standards, NZS1170:5 (2004) for structures with periods from 0.3s to 1.0s (~3-10 storeys) (Cousins and McVerry 2010).

The main-shock and numerous aftershocks, including several Mw 5+ aftershocks had resulted in widespread liquefaction failure and significant structural damage in particular with regards to unreinforced masonry (URM) structures. However, preliminary reconnaissance report of the seismic performance of the reinforced concrete (RC) buildings (Kam *et al.* 2010) indicated however low-to-moderate levels of damage for pre-1970s 'brittle' RC buildings, somehow contradictory to the expected level of damage given the levels of shaking and earthquake magnitude.

While the local and international communities attributed the lack of fatalities or widespread RC building collapses to excellent building codes and the fortunate timing of the main shock (Wood *et al.* 2010), it could have been argued that the characteristics of the ground motions in the CBD were not as demanding as the compatibility with the design spectra would suggest. The engineering and seismological communities are still investigating the characteristics of this rich set of ground motion records and its peculiar impacts on built environment (Cousins and McVerry 2010). In particular, the limited level of damage (structural and non-structural) in the pre-1970s RC buildings, contradictory to recent observation from other damaging earthquakes (Northridge, CA 1994, L'Aquila, Italy, 2009, Maule, Chile 2010), is a crucial question to understand. This paper, thus, seeks to provide some preliminary answers on the source of the 'good' performance of RC buildings of pre-1970s construction vintage and highlights some lessons for the recovery and reconstruction.

2 PRE-1970S RC BUILDINGS SEISMIC PERFORMANCE

2.1 RC Building stock in the Christchurch Central Business District (CBD)

The pre-1970s RC building stock within the Christchurch CBD, defined by the area surrounded by the four main avenues (Bealey, Dean, Moorhouse and Fitzgerald), was surveyed post-Darfield earthquake to complete a previously established inventory of pre-1970s RC building stock. Using the building data provided by Quotable Value New Zealand Ltd, Figure 1 shows the distribution of number of storeys and construction age of mid- to high-rise RC buildings in Christchurch and Wellington CBDs. Twenty-eight of 126 mid-to high-rise RC buildings in Christchurch CBD are of pre-1970s vintage and fourteen of the 28 are heritage-listed buildings. Figure 2 shows several examples of the typical pre1970s RC buildings within the Christchurch CBD.

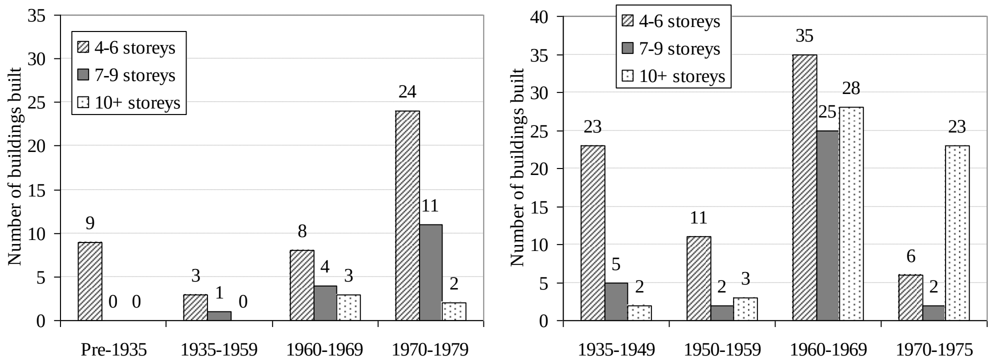

Figure 1. Buildings storey distribution for RC buildings above three storeys. a) Christchurch CBD (taken from (Kam *et al.* 2010), b) Wellington CBD (adopted from Blaikie and Spurr (1990)).

2.2 Distribution of damage

The seismic performance of pre-1970s RC buildings, as a whole, was generally very good. Some pre-1970s buildings showed some signs of incipient brittle failure modes such as onset of failure in masonry infill walls, column hinging and joint shear failures (Kam *et al.* 2010). Low-rise (one to two storeys) buildings performed very well. On the other hand, more than half of the twenty surveyed mid-rise to tall building suffered some levels of damage, more notable in flexible RC frames-only buildings. About 42% of the 65 surveyed RC buildings showed signs of minor damage, including cracks in the main structural elements (walls, beams, columns and joints) and infill panels.

Figure 2 shows a plot of the estimated building period (based on the building height and type of lateral resisting system) versus the estimated roof displacement (based on the observed damage including structural and non-structural components e.g. infills and glazing). While it is apparent that taller buildings lead to higher roof displacement, the trend in Figure 2 indicates that the more flexible buildings underwent higher displacement demand and suffered more damage. Stiff buildings with multiple walls (RC or infills) seem to perform relatively well. Nevertheless, the initial positive contribution of masonry infill walls on the pre-1970s RC buildings could have been negated if the infill panels had deteriorated.

Considering the higher density of occupancies of the RC buildings in New Zealand (for commercial and mixed-use), structural collapse or failure of these buildings can lead to a disastrous outcome in terms of life safety and financial losses. Furthermore, as evident from the demolition of the seven-storey Manchester Court of mixed URM and RC construction (Sachdeva 2010), in which a planned 6-weeks demolition is still uncompleted 18 weeks later, high-rise building collapse/failure can result in prolonged building and surrounding traffic/business downtime. Therefore, the high consequence of the seismic vulnerability of pre-1970s (or more recent) RC buildings cannot be understated.

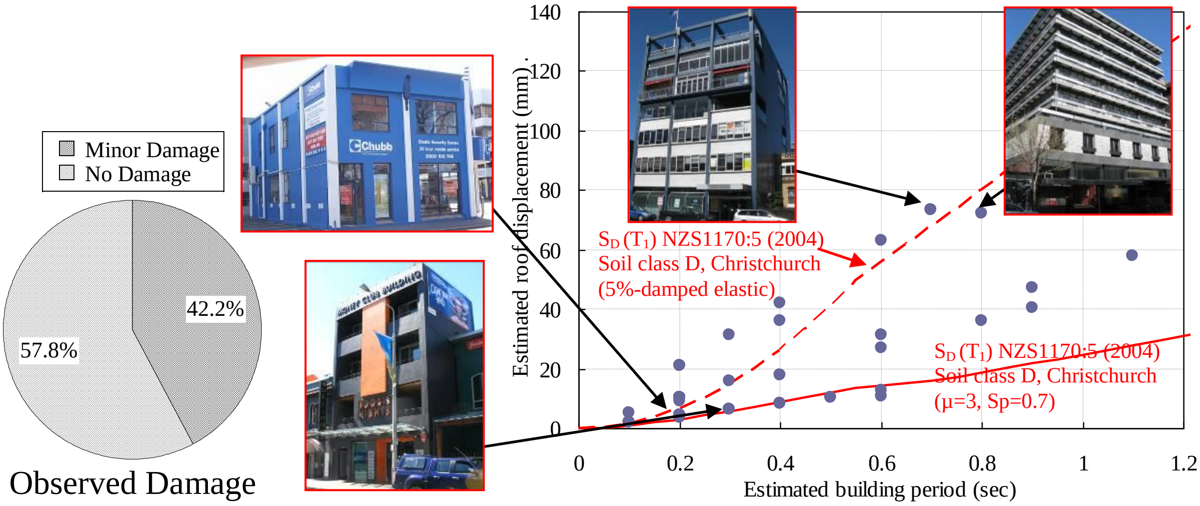

Figure 2. a) Percentage of damaged (minor to moderate) buildings in the surveyed pre-1970s RC buildings stock, b) Plot of estimated building period versus estimated roof displacement. The spectral displacement demands from the NZS1170:5 (2004) for Christchurch soil class D is shown as comparison.

3 COMPARISON WITH CODE INELASTIC DESIGN SPECTRA

The elastic response spectra (5%-damped) from four recorded ground motions (of both the principal and secondary horizontal motions) from the Christchurch CBD are compared with the site seismic design coefficient in Figure 3. The NZS1170:5 (2004) elastic design spectra for Christchurch site (Z/PGA=0.22g), distance R = 35km and soil class D (consistent with the four recording sites) is also plotted in Figure 3.

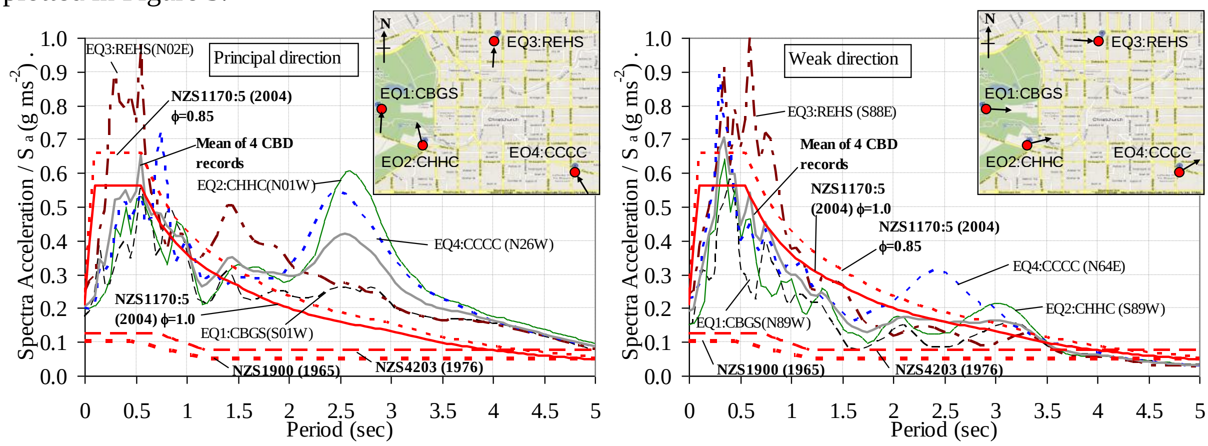

Figure 3. 5%-damped elastic acceleration response spectra in the Christchurch CBD and the NZS1170:5 design spectra (red solid) for Christchurch (soil class D, R=35km): a) Principal horizontal direction; and b) Weak horizontal direction.

The comparison of Figure 3a and Figure 3b shows a strong polarization in the long-period excitation (T_1>1.5s) in the North-South (principal) direction, which is approximately normal to the surface trace of the Greendale Fault rupture. This could be associated with forward directivity effects of the principal horizontal motions in addition to the site amplification factors (soft soil, basin effects, attenuation etc). Similarly the building damage distribution also suggested that high-rise buildings (and building contents) suffered higher damage in the North-South direction. However, it can be observed that at the low periods ($0.25s<T_1<0.5s$), typical of URM buildings, the direction polarity between the principal and weak directions diminished.

High acceleration demand (peaking around 2.5s) is evident in the four records in the principal direction. According to Cousins and McVerry (2010), this long period pulse corresponds to approximately 1000-year to 3000-year motions. For the periods 0.3s to 1.0s (for mid- and high-rise RC buildings), the four records spectra are approximately the design level motion (500-year return period). The elastic spectra alone do not fully explain the low damage observed in the Darfield earthquake.

The 'inelastic' response spectra, plotted in Figure 4, are generated by reducing the individual response spectrum using the NZS1170:5 inelastic reduction factors corresponding to medium ductility structures (μ=3 and Sp=0.7). Figure 4a compares the design lateral capacity (as designed without factoring any strength reduction factor, e.g. ϕ=0.85 for flexural inelastic action) for limited-ductility RC frames with the implied seismic action from this event. For clarity, the NZS1170:5 inelastic spectra with a 0.85 strength reduction factor is included in Figure 4a as the red short-dashed line.

Based on Figure 4a, the seismic demands for RC frames with limited-ductility were close to or below the NZS1170:5 design level (ϕ=1.0) for structures with periods between 0.1s and 1.4s. The higher spectral ordinate level for longer periods (T_1>1.4s) suggests that high rise buildings designed to the NZS1170:5 may have sustained significant seismic demand. If the ϕ=0.85 is considered, only the EQ3 (REHS) station exceeded the NZS1170:5 design spectra in periods between 0.1s and 1.4s.

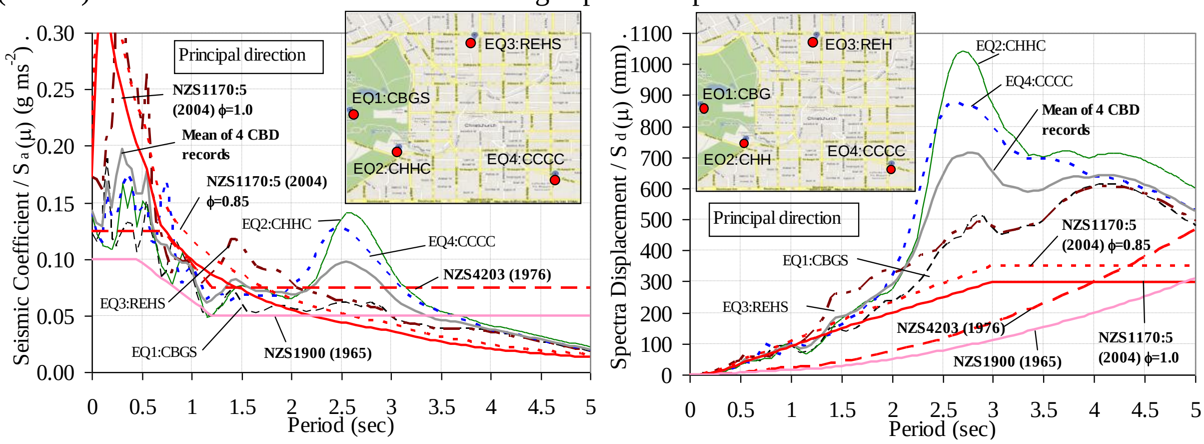

Figure 4. a) Inelastic acceleration response spectra (principal horizontal direction) in the Christchurch CBD and the NZS1170:5 design spectra (red solid) for Christchurch (soil class D, R=35km); b) elastic displacement response spectra and 5-% damped elastic design displacement spectra.

However, most pre-1970s building are not designed for the current NZS1170:5 (2004) design level. For better comparison, the seismic loadings for limited ductile RC frames according to the 1976 New Zealand Loading Standards (NZS 4203 (1976)) and the 1965 New Zealand Loading Standards (NZS1900(1964)) are also plotted on Figure 4. Both the NZS4203 and NZS1900 seismic loading have lower short period (T_1<0.5s) coefficients and higher long period (T_1>2.0-2.5s) coefficients when compared with the current NZS1170:5. Within the typical mid-rise RC building period range ($0.5s<T_1<2.0s$), NZS4203 and NZS1170:5 specify comparable seismic coefficient, while NZS1900 is approximately 33% lower.

The constant acceleration plateau of the older NZS4203 design spectra was exceeded in the short period range (T<0.65s) and in a range of long periods (T=2.2s to 2.9s). This suggests that mid- to high-rise ($2.2s> T_1>0.65s$) pre-1970s RC buildings which were designed to respond in-elastically up to μ=3 (limited ductility) may still have reserve capacity. On the other hand, low-rise (T_1<0.5s) pre-1970s RC buildings or buildings with significant masonry infills participation might induce higher seismic acceleration demand.

The seismic displacement demand implied by the elastic displacement spectra shown in Figure 4b, also suggests that the deformation demands on the pre-1970s buildings were generally low. For instance, the displacement demand, S_d ~ 100mm at effective/elastic period ~ 1.0s, suggests an inter-storey drift demand of less than 1.0% for a mid-rise 6-storey building. From the displacement-based design point of view, it seems that the NZS1170:5 design spectra do not adequately capture the long period seismic demand.

Lastly, it should be noted that the seismic code design spectra is a representation of an uniform hazard spectra –i.e. the elastic site spectra is derived from a probabilistic seismic hazard model based on a series of expected earthquake sources and distributed seismicity sources– distant or near-fault, high frequency or long period excitation. As such, it appears inappropriate to conclude that the M_w 7.1 Darfield earthquake has generated the expected design level shaking (in Christchurch CBD) and that the pre-1970s RC building stock does not need any seismic retrofitting/strengthening.

4 CASE STUDY NUMERICAL ANALYSES

4.1 Case study buildings and models description

A seven-storey building was selected from the Christchurch CBD buildings inventory as a numerical analysis case-study. Detailed damage survey and structural drawing investigation of this building after the Darfield earthquake were carried out. The non-linear dynamic responses of the case study building were investigated using Ruaumoko2D FE analyses (Carr 2008).

Lumped-mass and -plasticity modelling were adopted, where inelastic deformations were limited to discrete inelastic rotational springs in the joints and beams and columns. Two rotational joint springs with appropriate 'pinching' hysteresis rule which included cyclic strength and stiffness degradation were used to model the joint panel zone of existing RC frame as suggested by (Pampanin *et al.* 2003). Thin modified Takeda hysteresis (α=0.5, β=0) were used to model flexural hinges in beam and column elements.

The case study St Elmo Court building (Figure 5a) is a seven-storey non-ductile RC frame building with heavy masonry infill panels and exterior veneers. In the transverse direction (North-South orientation), the lateral resisting system consists of six five-bay RC frames, two of which had infill masonry. In the longitudinal direction (East-West orientation), six one-bay infilled RC frames and four seven-bay interior RC frames provide the lateral capacity of the building. For this study, only the transverse RC frame is considered.

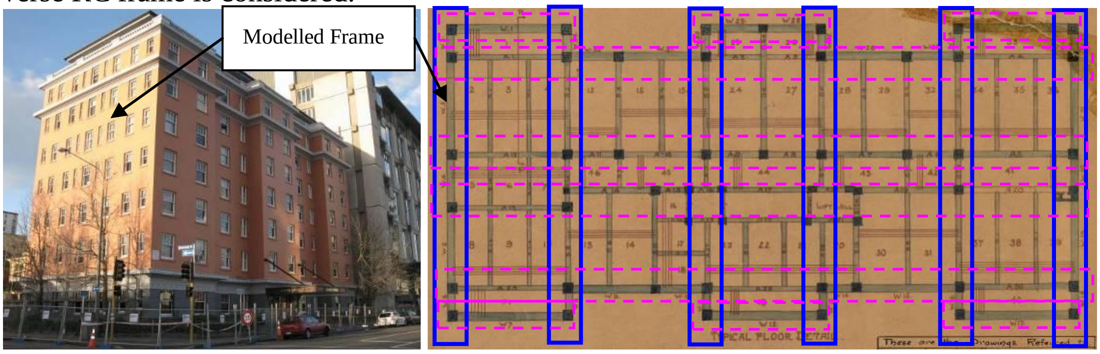

Figure 5. a) The South-West view of the St Elmo Court building; b) Typical floor plan of the St Elmo Court. Dark solid boxes and the light dashed boxes are the lateral load resisting elements in the transverse (N-S) and longitudinal (E-W) direction respectively.

4.2 Global deformation response results

The non-linear time history responses of the case study RC frame under two sets of strong ground motions are shown in Figure 6. The first suite of earthquake records is the four principal horizontal recorded records from the Christchurch CBD in the 4^{th} Sept 2010 event (as presented in the preceding section). The second suite of records consist of seven scaled 'near-fault' ground motions with forward directivity effects (see ref. (Kam 2010) for further details). The near-fault records are scaled according to the NZS1170:5 (2004) for the Christchurch seismicity (Z=0.22g, soil class D, R=10km, Sp=1)

Under the Darfield excitations, the RC frame performed relatively well, with maximum inter-storey drift demand of 0.75% to 1.25%. The result shows several columns at the levels 1 and 2 had shear cracks, with shear failure observed in EQ2 case. The double leaf infill panels are expected to be damaged up to level 4, due to the high deformation demand at the lower storeys. This corresponded well with the post-Darfield earthquake observation of the St Elmo Court building damage (Kam *et al.* 2010).

Despite the generally similar seismicity between the two suites of input records, significantly higher inter-storey drift responses were observed for the near-fault scenarios. Severe damage and possible soft-storey collapse are observed at level 2-3. The columns shear and the beam flexural capacities were exceeded at level 2-3. While the infill panels are not modelled in this simple analysis, it can be clearly seen that the infill panels will be significantly more damaged up to level 6.

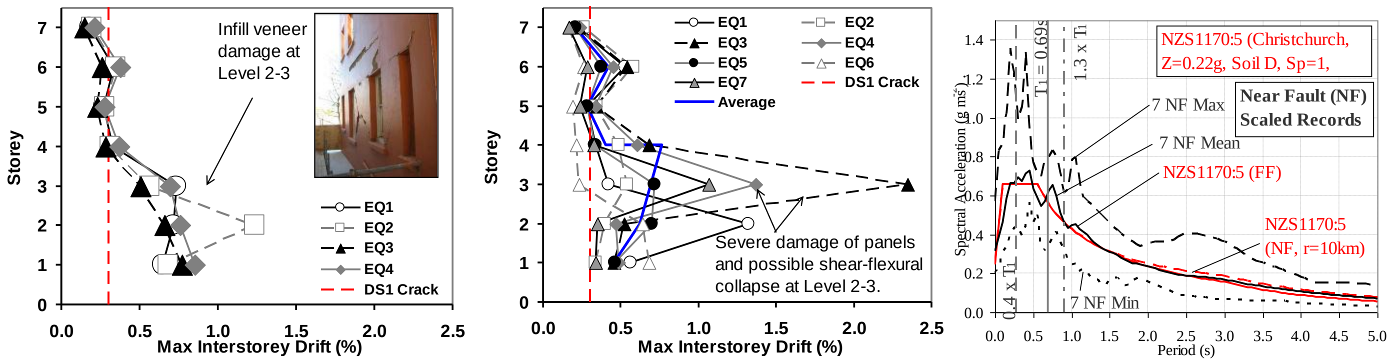

Figure 6. a) Inter-storey drift response under four 4th Sept Darfield CBD records; b) Inter-storey drift response under seven near-fault records scaled to the Christchurch site (Z=0.22g, Soil class D and Sp=1); and c) Response spectra of the scaled near-fault records, compared with the NZS1170:4 (2004) spectra for Christchurch site.

5 EXPERIMENTAL CONFIRMATION

A shaking table test of a 3-storey RC frame 40%-scaled model (Figure 7b), representative of the pre-1970s construction practice was carried out to understand the seismic vulnerability of these buildings (Quintana-Gallo *et al.* 2011). In the first stage of testing, column lap-splice failures (and soft-storey) at the 3rd floor dominated the response. In the second stage of testing, the repaired test frame (without lap-splice deficiency) was tested using an actual record from the Darfield 2010 earthquake and an actual record from the Maule Chile 2010 earthquake. The Maule record is an example of a possible sub-duction zone high magnitude but long distance event (such as the Alpine Fault scenario, a hazard of which the likelihood has not diminished despite the Darfield earthquake). Figure 7a shows the input motion acceleration time-histories and the response spectra of the second stage of testing.

The test result clearly highlighted the brittle 'on-off' (e.g. either elastic behaviour or brittle failure) seismic behaviour of the non-ductile pre-1970s building. Under the Darfield CHHC input (PGA=0.22g, distance = 35km), the recorded inter-storey drifts were less than 1% in all floors, with near elastic response. Minor flexural cracks were observed in the beams and columns (Figure 7c top). Under the Maule Marga-Marga input (PGA=0.34g, distance = 290km), severe damage was observed at the 1st floor beam-column joints. Maximum recorded inter-storey drifts of 4.0%, 2.5% and 1.0% were observed in the 1st floor, 2nd floor and 3rd floor respectively.

The experimental results also highlight the limitation of lateral strength (instead of displacement demand) as the engineering design parameter, considering the brittle nature of these non-ductile RC frames. Secondly, the test results also confirm that the lower-than-expected damage observed in the Darfield earthquake is possibly a consequence of the low-frequency, short duration and low number of cycles of excitation.

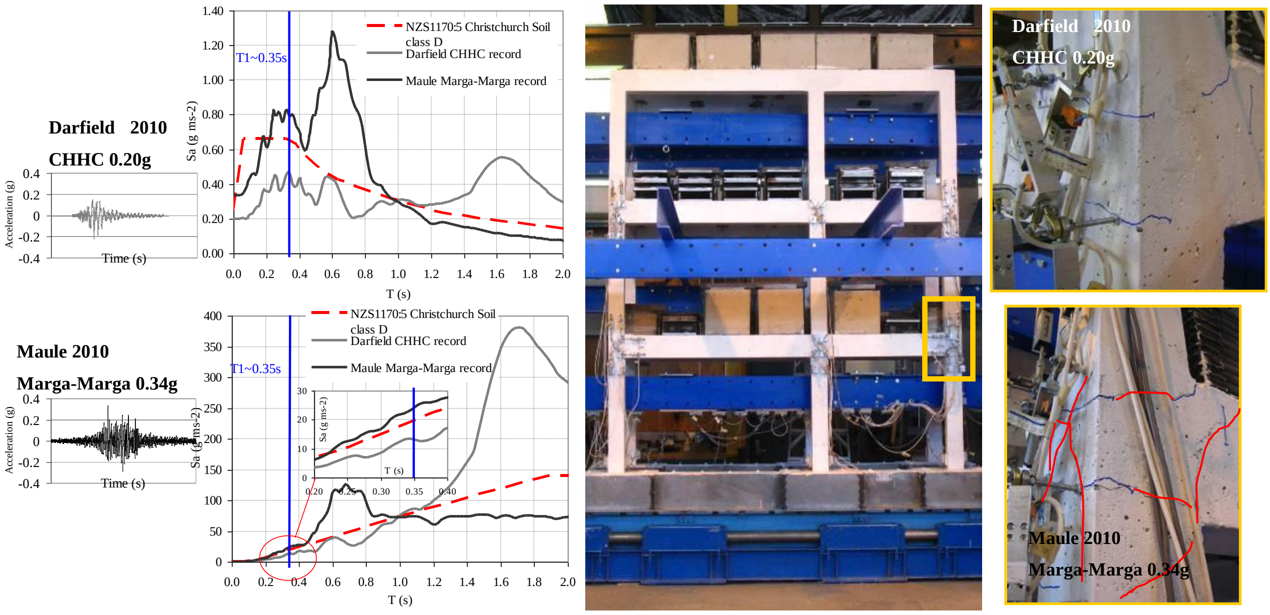

Figure 7. Shaking table test of three-storey non-ductile pre-1970s RC frame building (Quintana-Gallo *et al.* 2011). a) Input motion and response spectra in the model domain; b) The front elevation of the test frame model; and c) top: First storey beam-column joint after the scaled Darfield CHHC record, and bottom: First storey beam-column joint after the scaled Maule Marga-Marga record.

6 CONCLUSIONS

The 4th Sept 2010 M_w 7.1 Darfield (Canterbury) earthquake has been inappropriately considered as the 'design level test' of the seismic performance of New Zealand urban RC buildings since the introduction of modern seismic codes in the mid-1970s. While the widespread liquefaction and lateral spreading ground failures has been the dominant contributor to losses and damages, the high consequence of widespread damage and collapse of mid- to high-rise building stock of RC buildings cannot be understated.

As shown by the effects of the different input signals on the numerical responses and the shaking table responses of pre-1970s RC buildings in the preceding sections, the relatively good performance of the pre-1970s RC buildings can be negated in different seismic scenarios due to their inherent brittle 'on-off' failure mechanism. This paper presents several hypotheses, supported by some numerical and experimental evidences, on why the pre-1970s buildings performed well in the Darfield earthquakes for further studies and consideration:

a) The influence of the ground motion characteristics such as the short duration, general lack of energy in the low-medium period / medium-high frequency range, and filtering effects due to very soft subsoil of Christchurch City.

b) The initial positive influence of masonry infills or 'non-structural' walls in stiffening the buildings, reducing their fundamental periods (and reducing the force demands in this particular earthquake), and increasing the lateral strengths.

c) The influence of directivity and polarity of the ground motions (which was stronger in the North-South direction).

All the above might have helped in not triggering the aforementioned switch on-off mechanism which would have likely led to catastrophic brittle failure/collapse, as typically observed for pre-1970s RC buildings in past earthquake events. The brittle failure modes and the high vulnerability of pre-1970s RC building, as evident from the numerical studies and recent shaking table tests from the University of Canterbury, are good reminders of the seismic risks to the practitioners and society. Making the mistake of taking for granted the relatively good performance of this vulnerable building stock under a specific input ground motion can be fatal and costly in other earthquake scenarios.

7 *PRELIMINARY OBSERVATION FROM THE 22ND FEB 2011 M_W 6.3 EARTHQUAKE*

Note: at the time of writing and review of this manuscript, the 22nd Feb 2011 M_w 6.3 Christchurch earthquake has occurred approximately 10km from the Christchurch CBD region. Unlike the Darfield earthquake, this lower magnitude but significantly higher intensity of ground shaking has resulted in severe damage/collapse of RC buildings. In particular, a 1960s building consisting of RC frames and core-wall has fully collapsed (Figure 8a), resulting in loss of human lives. Severe column, beam-column joints, walls and beams damage have been observed in RC buildings built in the 1950s to the 1980s (Figure 8b-f). While it is premature to make any significant conclusion of the observed structural failures and damage in the 22nd Feb event, these high-impact consequences of this catastrophic event have unfortunately reaffirmed the conclusions above (stated and written prior to the 22nd Feb earthquake).

Figure 8. Preliminary observations from the 22nd Feb 2011 M_w 6.3 Christchurch Earthquake: a) The collapse of the 1960s Pyne Gould Corp building; b) Column and joint shear damage at a 9-storey RC building; c-d) Failure of critical wall and column elements resulting in significant tilt of the Grand Chancellor building; e) Compressive crushing failure of shear wall; and f) Beam-column joint failure at the 1st floor connections.

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