

SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS IN THE SEPTEMBER 2010 DARFIELD (CANTERBURY) EARTHQUAKE

Weng Y. Kam¹, Stefano Pampanin², Rajesh Dhakal³,
Henri P. Gavin⁴ and Charles Roeder⁵

SUMMARY

This paper describes observations of damage to reinforced concrete buildings from the September 2010 Darfield (Canterbury) earthquakes. Data was collated from first-hand earthquake reconnaissance observations by the authors, post-earthquake surveys, and communications and meetings with structural engineers in Christchurch. The paper discusses the general performance of several reinforced concrete building classes: pre-1976 low-rise, pre-1976 medium rise, modern low- and mid-rise, modern high-rise, industrial tilt-up buildings, advanced seismic systems and ground-failure induced damaged and retrofitted RC buildings. Preliminary lessons are highlighted and discussed. In general, reinforced concrete buildings behaved well and as expected, given the intensity of this event.

1 INTRODUCTION

1.1 General

This paper focuses on the seismic performance of reinforced concrete (RC) buildings in the 4th September 7.1M_w main event and the subsequent aftershocks. Buildings designed before and after the adoption of seismic-resistant design codes were examined.

No RC building collapsed during the earthquake. At the time of writing (almost two months after the event), only 10 out of the 717 RC buildings inspected remained classified as “red tag” (deemed “unsafe” in the Building Safety Evaluation (BSE) assessment [1]). Fifty-five RC buildings were assigned a “yellow tag”, which means they were available for restricted use only.

The apparently good seismic performance of different classes of RC buildings offers an opportunity to reflect on the New Zealand RC design practice and to derive a number of preliminary lessons from the earthquake.

1.2 Imposed Seismic Loads

While the 7.1 M_w earthquake was close to the largest expected magnitude for a rupture near the city of Christchurch, the ground shaking intensity, in terms of the seismic response spectra, was comparable to the inelastic design spectra for Christchurch (Z = 0.22g, soil class D, R = 35 km) according to the 2004 New Zealand Loading Standards (NZS1170:5)[2]. A 7.1M_w earthquake event, 40 km from Christchurch, contributes approximately 15% of the seismic hazard with a 475-year return period [3].

“Inelastic” response spectra from four recorded ground motions (principal direction) from the Christchurch Central Business District (CBD) are compared with the site seismic design coefficient in Figure 1. The NZS1170:5 design spectra and the record spectra were reduced using the NZS1170:5 inelastic reduction factor corresponding to medium ductility structures ($\mu=3$ and $S_p=0.7$).

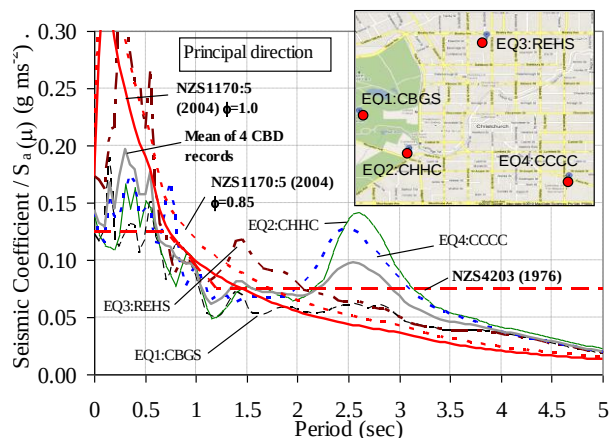


Figure 1: Comparison of inelastic spectra of four records in the Christchurch CBD and the NZS1170:5 design spectra (red solid) for Christchurch (soil class D, R = 35 km), reduced assuming limited-ductility RC frames ($\mu=3$ and $S_p=0.7$).

Figure 1 compares the design lateral capacity (as designed without factoring any strength reduction factor, e.g. 0.85 for

¹ Graduate researcher, Dept of Civil and Natural Resources Eng, Uni. of Canterbury. weng.kam@pg.canterbury.ac.nz

² Associate Professor, Dept of Civil and Natural Resources Eng, Uni. of Canterbury. stefano.pampanin@canterbury.ac.nz

³ Associate Professor, Dept of Civil and Natural Resources Eng, Uni. of Canterbury. rajesh.dhakal@canterbury.ac.nz

⁴ Associate Professor, Dept of Civil and Environmental Eng, Duke Uni. hpgavin@duke.edu

⁵ Professor, Dept of Civil and Environmental Eng., Uni. of Washington, Seattle. croeder@u.washington.edu

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 1 as the red solid line. The seismic loading for limited ductile RC frames according to the 1976 New Zealand Loading Standards (NZS 4203 [4]) is also plotted on Figure 1 as a red dashed line. Detailed retrospective comparisons of New Zealand loading standards have been published by Davenport [5] and Fenwick and MacRae [6].

Based on Figure 1, the seismic demands for RC frames with limited-ductility were close to or below the NZS1170:5 design level for short period structures ($0.1 \text{ s} < T < 1.4 \text{ s}$). The higher spectral ordinate level for longer periods ($T > 1.4 \text{ s}$) suggests that high rise buildings designed to the NZS1170:5 may have sustained significant seismic demand. The constant acceleration plateau of the older NZS4203 design spectra was exceeded in the short period range ($T < 0.65 \text{ s}$) and in a range of long periods ($T = 2.2 \text{ s}$ to 2.9 s).

2 CHRISTCHURCH RC BUILDING STOCK

2.1 Buildings Distribution and Types

Christchurch has a mix of newer RC buildings, with modern detailing, and older non-ductile RC structures.

Using the building data provided by Quotable Value New Zealand Ltd, Figure 2 illustrates the distribution of number of storeys and construction age of mid- to high-rise RC buildings in Christchurch. 126 mid-to high-rise RC building were identified within a set of 736 ‘concrete’ buildings. Twenty-eight of these mid-to high-rise RC buildings are of pre-1970s vintage, fourteen of which are heritage-listed building.

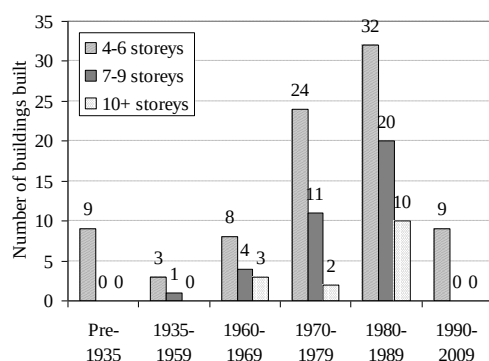


Figure 2: Buildings storey distribution for RC buildings within the City Centre and above three storeys (source: Quotable Value NZ Ltd).

A study carried out in the 1970s listed about 40 non-ductile RC buildings (both non-heritage and heritage) in Christchurch CBD [7]. A recent review [8] of the heritage buildings within the Christchurch City Council (CCC)’s City Plan shows that out of the 490 heritage-listed buildings, 29 are non-ductile. Fifteen of these are low-rise buildings with less than four storeys and fourteen are buildings with four to six storeys. Twenty-one of these twenty-nine buildings were built in the intra-war period of 1920-1939.

Two weeks after the main shock, a detailed damage survey was carried out by a team of researchers of the Foundation for Research, Science and Technology (FRST) Seismic Retrofit Project [9] from the University of Canterbury. A previously established inventory of pre-1970s RC buildings was expanded with an additional dataset and damage observations. Figure 3 presents some of these findings.

About 42% of the 65 surveyed RC buildings showed signs of minor damage, including cracks in the main structural elements and infill panels. Clay brick and concrete block infill

walls were commonly found in most pre-1970s RC frames buildings, mostly without any separation or seismic detailing.

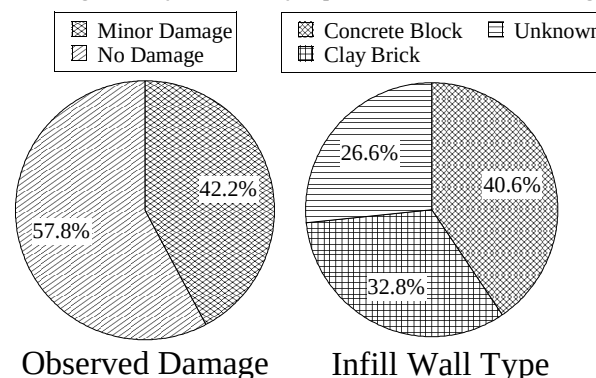


Figure 3: Observed damage in pre-1970s RC buildings stock and type of infill walls.

2.2 Building Safety Emergency Tagging Distribution

Within 48 hours from the main event, emergency response teams of structural engineers and local authorities carried out the Building Safety Evaluation (BSE) procedure (i.e., tagging) [1] under the state of emergency authority. The NZSEE BSE guidelines draw heavily from the ATC-20 Post-earthquake Safety Evaluation of Buildings procedures [10]. Table 1 summarises the distribution of BSE tagging of the 717 inspected RC buildings within the Christchurch City Council boundaries ($1,493 \text{ km}^2$), as on the 20th Sept 2010.

It is worth noting that the apparently high number of the yellow tagged buildings was a result of non-structural damage which resulted in reduced access and health and safety related restriction in their use. A number of red tagged buildings were also due to land damage (liquefaction), rather than direct shaking-induced damage.

Table 1. Distribution of Building Safety Evaluation tagging of all RC buildings (source: CCC). The tag colours are as defined in the NZSEE BSE guidelines [1].

Types of Constructions	NZSEE Building Safety Evaluation Tagging		
	Green	Yellow	Red
Reinforced Concrete (RC) Frames	270 (89.7%)	29 (6.5%)	2 (0.7%)
RC Shear Wall	92 (92%)	6 (6%)	2 (2%)
RC Frames With Masonry Infill	132 (90.4%)	10 (6.8%)	4 (2.7%)
Tilt Up Concrete	158 (92.9%)	10 (5.9%)	2 (1.2)

Practising engineers have reported a large variation in the evaluation criteria used during the initial BSE rapid assessment phase. It was found that in the initial stage, many RC frames buildings with signs of plastic hinges and substantial concrete cracking were given yellow or red tags. As plastic hinges in concrete structures are acceptable, reasonable post-event outcomes, subsequent and more detailed seismic assessment have changed the BSE status of many of these buildings to “green” tags.

Practising engineers have also appreciated the “live” status of BSE tags, as new information was revealed with further inspections and additional aftershocks. With several moderate magnitude but shallow and near-by aftershocks occurring, several buildings that had been initially green-tagged were given “restricted use” yellow tags by the Council pending further structural evaluation.

With the end of the state of emergency, any change in BSE status required council approval, in line with section s124 of the New Zealand Building Act of 2004 [11].

3 GENERAL PERFORMANCE OF RC BUILDINGS

RC buildings generally behaved well, as expected, given the intensity of this event. For many RC buildings, no apparent structural damage was observed. Minor structural damage including column and beam flexural cracks and joint/wall shear cracks were observed in a number of RC buildings. Non-structural damages observed included stairway-structure interaction, broken windows, cracking of the infill brick panels and fallen false ceiling and plaster works. In the following discussion, the classification “pre-1970s” and “modern buildings” will refer to buildings designed prior to and after the 1976 “modern” seismic code NZS4203 [4], respectively.

3.1 Pre-1970s low-rise RC buildings

RC buildings designed and built prior to the introduction of the modern principles of capacity design, were predominantly single or double storey low-rise commercial buildings. Many of these buildings (about 82% of the surveyed CBD 1935-70s building stock) are regular in plan.

Prevalent low-rise construction involved RC frames with solid clay bricks or hollow concrete blocks infill. As compared to many older, unreinforced masonry buildings, masonry infill walls used in these pre-1976 RC frames sustained relatively little damage. Many of the infill walls were in good condition prior to the earthquake. Reinforced masonry infill walls (which had been tested as early as 1932 [12]) and RC block masonry infill walls were widely used by early 1960s [13], following the 1955 NZS95 Model Building By-Laws [14].

Figure 4 shows an example of a typical low-rise 1950s-60s RC frame with concrete blocks masonry infill walls. Separation cracks between the structural frames and the infill walls were observed. No diagonal shear or horizontal-sliding cracks were observed in the masonry infill walls. Flexural cracks were observed in several beams and columns.



Figure 4: *Low-rise pre-1970s RC frames with masonry infills. Separation cracks of the infills and RC frame were observed. (Photo credit: Weng Y. Kam).*

Figure 5 shows two more examples of low-rise RC buildings with more noticeable damage in the structural frame elements. Figure 5a-b shows a three storey RC frame building with clay brick infill walls in the perimeter frame, built in the 1950s-60s. Flexural cracks at the beam ends and diagonal cracks on the masonry were observed. Figure 5c-d presents an example of low-rise RC frames with observable joint shear cracking.

These low-rise RC buildings are generally stiff, particularly if the masonry infill walls act in-plane with the RC frames. The natural period for RC frame structures with intact infill walls is significantly shorter as compared to that of bare frames. RC frames with intact infill walls therefore attract higher seismic forces. Testing of non-ductile infilled RC frames (e.g. [15]) shows that the participation of solid clay infill can provide an

over-strength of up to 1.5 to 2.5 times that of bare frames. Considering such additional capacity to the conventional 0.1-0.12g lateral design in the early loading codes [14], and the fact that the demands from this earthquake were less than the design code demand, the observed low-level of damage of the low-rise buildings is not surprising.

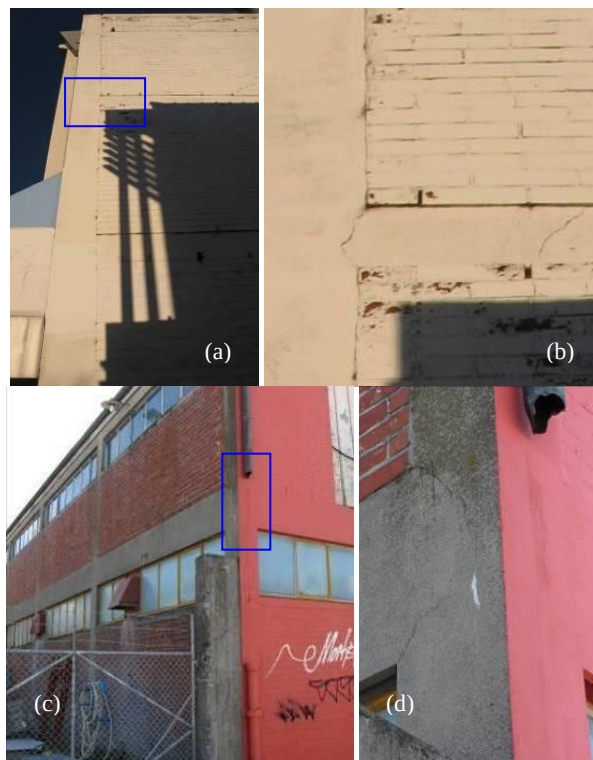


Figure 5: *Visible residual cracks in the beam, column and joint elements in low-rise pre-1970s RC frames with masonry infills. (Photo credit: Weng Y. Kam).*

3.2 Pre-1970s mid- to high-rise RC buildings

The data presented in Figure 2 and collected in the post-earthquake survey lists about 25 to 30 mid- to high-rise pre-1970s RC buildings in Christchurch. Many of these structures suffered very little to minor damage. Figure 6 shows two examples of RC buildings built in the 1960s that have shown no visible structural damage.

The first is a six storey building with an irregular plan built of hollow RC masonry blocks. The 150 mm outer and 100 mm inner leaves are reinforced with $\frac{1}{2}$ " (12.7 mm) and $\frac{3}{8}$ " (9.5 mm) rods at 24" (~600 mm) centres, respectively. Floors are 7" RC flat plates. The exterior and interior paints maintain good mortar strength and masonry condition. Given the structural type this building is arguably an example of Reinforced (Concrete) Masonry.



Figure 6: *Mid-rise pre-1970s RC buildings with no apparent damage. (Photo Credit: Weng Y. Kam).*

The second building is the eleven storey former Government Life Building. Built in 1965, it was the tallest in Christchurch at the time. It is a four-bay by seven-bay RC frame with 20" (~500 mm) RC square columns and 150 mm thick RC lift-core walls. No apparent structural or non-structural damage was observed and the building was occupied soon after the earthquake.

However, there are at least five mid- to high-rise pre-1970s RC buildings which suffered moderate-to-severe damage to the structural and non-structural elements (known at this stage to the authors).

Figure 7 shows the St Elmo's Court building, a heritage-listed 1930 art-deco styled seven storey former apartment block. It has a series of gravity-designed RC frames coupled with 150 mm thick RC staircase core walls. The core walls had minor diagonal and flexural cracks up its elevation. The columns varied in size up the elevation, with typical sizes of 22"-18"-15" (560 mm – 460 mm – 380 mm) squares. One of the exterior columns had a clear shear failure (Figure 7c). Horizontal cracks along the floor level and below the beam soffits were observed in most storeys, indicative of inter-storey deformation.

There is a double-width solid-clay bricks veneer along the perimeter frames. Shear diagonal cracks were observed on the larger exterior panels (Figure 7b). A closer inspection revealed good concrete mortar quality on the infill walls. The failure on the infill walls were predominantly the fracture of solid-clay bricks (Figure 7d). Most of the interior brick infill walls had been removed during a refurbishment in 1980s. Interestingly, some of the preserved original 1930 plastered timber infill panels show no apparent damage (Figure 7e).



Figure 7: *St Elmo Courts seven storeys RC buildings. Shear cracks observed in the masonry infills and column. (Photo Credit: Weng Y. Kam and Stefano Pampanin - (c)).*

Figure 8a shows an eight storeys RC frame-wall building built from 1962 to 1966 at the University of Canterbury Ilam campus. The building has a rectangular plan with a large aspect ratio. The RC frames in the weaker direction were damaged. Cracks in the beams and walls were observed. For example, in the top floor, beam residual cracks up to 5 mm

wide indicated significant yielding of steel reinforcement, (Figure 8b-c). The building also experienced significant non-structural damage including wall partitions cracking, ceiling damage, broken windows, damage at seismic separation, and toppling of containers containing chemicals (e.g. paraffin oil).



Figure 8: *Beam plastic hinge of 1960s mid-rise RC building. (Photo Credit: Rajesh Dhakal).*

3.3 Modern low- and mid- rise RC buildings

Very little damage in modern low-rise RC buildings has been reported, apart from damage due to liquefaction. Low-rise RC building construction using frames or shear walls (except for industrial tilt-up panel building) is uncommon beyond the 1970-80s. The modern low-rise buildings inspected generally exhibited no sign of structural damage.

Mid-rise buildings with RC ductile/nominally ductile frames or walls are relatively popular forms of construction. Very few modern mid-rise (four to nine storeys) RC buildings were reported to have suffered structural damage. Content and non-structural damage was cited as the reason for some of these buildings to be evacuated and yellow or red-tagged. Figure 9a shows an undamaged example of this building class.

In several mid-rise RC buildings with cast-in-situ shear walls, diagonal and horizontal cracks were observed in the thick RC shear walls. Figure 9b shows an example of such minor damage, observed in a five storeys building with heavy shear wall lateral system.



Figure 9: *a) Modern mid-rise RC frames building; b) Shear diagonal cracks on modern thick shear walls. (Photo credit: Weng Y. Kam).*

3.4 Modern high-rise RC buildings

Damage in modern tall RC buildings was consistent with the expectations of an event having 1/3 to 2/3 of the NZS1170:5 Ultimate Limit State (ULS) design excitation. Evidence of plastic hinges and inelastic response was apparent in some buildings. The observed performance was closer to an Immediate Occupancy limit state than to a Life Safety or Ultimate limit state, indicating either a superior performance of the RC buildings, a lower than expected input shaking intensity, or a combination of both.

As Figure 1 suggests, the seismic demand at the long period ($T > 1.5$ s) exceeds the 'inelastic' lateral design capacity that many of these structures were designed for, regardless of whether their designs were based on the older NZS4203 or the current NZS1190:5 (for $\mu = 3$). High rise RC frame buildings with more than ten storeys could have been particularly susceptible to this long period spectral acceleration amplification.

There are approximately ten buildings with ten or more storeys, constructed between 1980 and 1989, as shown in Figure 2. Of these, at least six (known to the authors) exhibited evidence of moderate damage. Several of them were under restricted use (yellow-tagged) and required further repair two months after the earthquake. The lateral systems of these structures are generally a mix of frames, walls and dual-systems.

Figure 10 shows a fourteen storey commercial building with RC shear walls around the staircase/escalator core, with coupled-link beams to the frame systems. The perimeter frame was designed primarily for gravity loading only. Evidences of concrete spalling, exposing column and beam reinforcement, were observed in many beam-column connections throughout the building elevation.

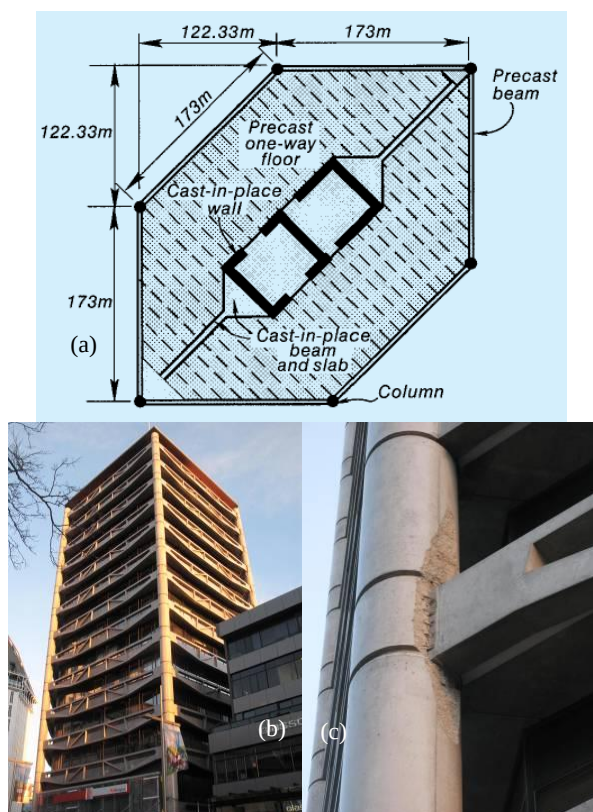


Figure 10: Fourteen storeys RC core walls building with perimeter gravity frames: a) Structural layout [16]; b) Damaged gravity frame. (Photo credit: Weng Y. Kam).

Figure 11 shows an example of a twelve storey two-way RC frame building, built in the 1980s. The building was initially green-tagged but was subsequently yellow-tagged and evacuated following an aftershock three days after the main event. Cracks on the main structural elements grew larger with each of the numerous moderate aftershocks ($M_w = 4.5-5.4$).

Indications of plastic hinges in the beams at the exterior bays were evident in the second to seventh storey. The yielding of the reinforcing bars was up to three times the yield strains, estimated from the 3-4 mm wide residual crack width. Cracks ran throughout the depth of the beam suggesting yielding of both the top and bottom bars.

The yielded beams were part of the frames in the north-south (NS) direction, suggesting that the shaking was dominated by the NS component. The plastic hinges existed only in the outermost bay; in the inner bays the beams had fine cracks but not to a level that would suggest yielding of reinforcing bars.

The building had prestressed precast concrete double-tee units spanning in the NS direction parallel to the damaged frames. Cracks between the precast floor units and the transverse beams (east-west) indicated some level of beam elongation effects from the plastic hinges.



Figure 11: Twelve storey RC frame building beam plastic hinges. (Photo credit: Rajesh Dhakal).

Practising engineers reported that many of these modern mid- to high-rise buildings deformed to 1/3 to 2/3 of their design level. As significant portion of the high-rise buildings were designed in the 1980s following the 1976 NZS4203 loading standards, these buildings can have substantially higher design strength when compared to the current NZS1170:5 standards. Detailed inspections of these mid- to high-rise RC buildings will be necessary in order to comprehensively evaluate damage and performance of these structures.

3.5 RC Tilt-up industrial/commercial buildings

Precast concrete tilt-up panels in conjunction with steel/timber roofs and/or steel portal frames is a popular construction type for single storey industrial/commercial buildings in New Zealand. As listed in Table 1, twelve tilt-up panel buildings were still considered unsafe (yellow/red tagged) in the CCC territory.

Many of these tilt-up panel buildings were damaged by liquefaction failure. Figure 12 shows a supermarket building in the Kaiapoi town (17 km north of Christchurch City), that is scheduled to be demolished due to liquefaction damage. It consists of tilt-up precast concrete panels and portal steel frames, founded on flat RC slab on-grade.

Practising engineers have reported moderate anchorage failure of the shear connectors. Some out-of-plane tilting of panels attributed to bad top connector detailing have also been reported. In many cases, the critical transfer between the roof diaphragm and panels was found to be in good condition.



Figure 12: A tilt-up precast concrete with steel portal frames supermarket to be demolished due to differential ground settlement from liquefaction. (Photo credit: Weng Y. Kam).

3.6 Advanced seismic resisting RC systems

The Darfield (Canterbury) earthquake has also tested a few innovative advanced seismic resisting RC systems such as the post-tensioned hybrid (self-centering/dissipating) jointed-ductile RC (PRESS) technology building.

The newly constructed four storeys Southern Cross Hospital's Endoscopy Consultant Building is the first South Island PRESSS-technology building, incorporating jointed-ductile connections (described in Appendix B of the 2006 NZS3101 [17] and the PRESS Design Handbook [18]). The lateral system in the NS direction is made up of four precast concrete unbonded post-tensioned frames with top-only bonded mild steel at the beam-column connection. In the EW direction, 250 mm thick precast concrete unbonded post-tensioned coupled-walls with U-shaped flexural plates (UFPs) were used. Figure 13 shows the East elevation of the building and a close-up of one of the interior beam-column joint.

The post-tensioned frames and coupled walls were expectedly undamaged. Vertical cracks along the grout pad at the beam-column interface suggested that the 'rocking' mechanism of the frame system was indeed activated. However no residual crack width was noticeable in the beam-plastic hinge, due to the designed re-centering action of the unbonded post-tensioned tendons. Several of the gravity corbels had hairline to 0.5 mm cracks confirming a not-negligible rotation-induced compression force (plus seismic shear) on these corbels. Hairline cracks were also observed in the secondary transverse gravity frame in the EW direction. Negligible non-structural (one single glass panel) and content damage was reported.



Figure 13: Self-centring precast concrete system implemented for a newly constructed private hospital facility. (Photo credit: Weng Y. Kam).

3.7 Ground failure induced damage

Liquefaction and lateral sliding ground failures were a major source of structural damage, particularly to residential houses. In many of these areas, RC buildings also suffered significant damage.

Figure 14 shows an example of a two storey RC building constructed in the mid 1960s along the Avon River. This building had sustained significant damage because of lateral spreading of the soil under the foundations of the columns. The column line adjacent to the river was the only column line with apparent soil movement. Most damage was concentrated in the beam-column joints.

In Figure 15, severe differential settlement (up to 300 mm at the south end) resulted in significant tilting of the three storey RC frames superstructure. No damage was observed on the superstructure from exterior inspection.



Figure 14: Lateral spreading and liquefaction induced displacement demand resulting in beam-column joint failure. (Photo credit: Charles Roeder).



Figure 15: *Liquefaction failure-induced tilting and differential settlement of a three storeys RC frames building built in 1978 at Kaiapoi. (Photo credit: left: Rajesh Dhakal, right: Weng Y. Kam).*

4 PERFORMANCE OF RETROFITTED RC BUILDINGS

4.1 General

Currently, there is no active registry of seismically-retrofitted buildings in Christchurch. As such, despite the numerous examples of retrofitted RC buildings in Christchurch, it is difficult to systematically study the performance of these buildings. Structural design firms inspected their own retrofit designs after the earthquake. Discussions with engineers at these firms suggest that most retrofitted RC buildings performed very well, with no unexpected damage.

4.2 Case-study examples

The two-storey former Physical Sciences Library at the University of Canterbury Ilam campus just undergone seismic retrofit work a few days before the 4th September earthquake. Figure 16a shows an archive photograph of the building in 1968 when it had just been completed by the New Zealand Ministry of Works. The lightweight roof is supported by interior columns and the upper floor is supported on four pairs of two-way RC frames. The stirrups in the as-built columns are unclosed and insufficient for ductile behaviour at the potential plastic hinge zones.

The seismic retrofit strategy was to increase the ductility of the ground floor frames. The 380 mm diameter ground floor columns were identified as the weakest link of the superstructure. Fibre-reinforced polymers (FRP) jacketing was used to confine the plastic hinge zone of the columns (Figure 16b-c). Two layers of SikaWrap-100 glass fibre fabric wrap with Sikadur-300 epoxy were used on 600 mm lengths at the top and bottom of the columns.

After the earthquake, flexural cracks were observed in the plastic hinge regions within the FRP wrapping. The columns effectively formed top and bottom flexural hinges as the lateral resisting mechanism. No noticeable residual deformation of the building was noted.

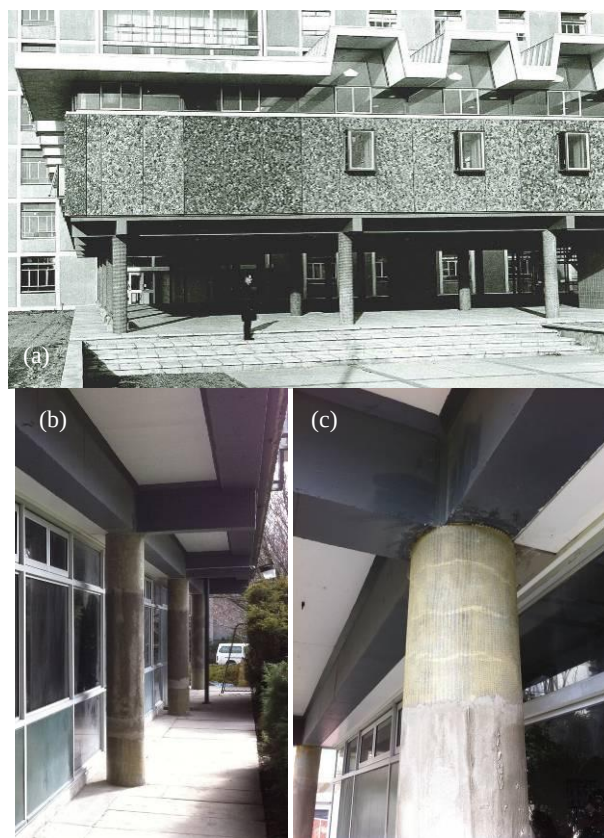


Figure 16: *Column retrofit using FRP jacketing for the two storeys Sciences library building. (Photo credit: a) UoCanterbury archives, b-c) Grant Wilkinson of Ruamoko Solutions).*

Figure 17 shows attempts to secure the brick wall parapets on a single storey 1950s-era infilled RC frame building. While none of the parapet failed during the 4th September earthquake, growing cracks along the infill panels and interior damages from the aftershocks resulted in the closure and 'red-tagging' of the structure (an emergency medical facility) five days after the main event.

Figure 18 shows another successful retrofit example of a 1950s three storey RC frame building with non-ductile detailing. The seismic retrofit strategy involves global strengthening by additional RC shear walls and local ductility capacity upgrading by confining the as-built non-ductile columns with FRP wraps. The building performed very well during the earthquake. No structural or non-structural damage was observed from an exterior-only inspection.



Figure 17: *Infill walls parapet retrofit for infilled RC frames (Photo credit: Weng Y. Kam).*



Figure 18: Seismic retrofit of non-ductile RC frames with FRP jacketing of the columns and with additional new shear walls. (Photo credit: Weng Y. Kam).

4.3 Seismic retrofitting policy impact

Six days after the main event, the CCC's councillors unanimously passed the Earthquake-Prone Buildings (EPB) policy [19], which had been under its 5-year cycle review. The EPB policy establishes a time frame of 15-30 years for the earthquake strengthening of EPB (that do not meet the 33% of the current building code requirements) to a 67% of the current building codes requirements. The new policy also covers those earthquake-damaged buildings applying for building consents for repairs activities (Section 2.3.6 of [19]).

Preliminary feedback from engineers and owners indicated that while the new EPB policy is good for the overall seismic resilience outcome, it resulted in higher-than-anticipated repair and retrofit costs as a consequence of the Canterbury earthquake. As a result, there has been, and could continue to be, a tendency for owners to demolish their earthquake-damaged buildings rather than to repair and seismically upgrade them. Building insurers have no statutory requirement to pay for the seismic upgrade to the 67% requirement; the cost of these mandated upgrades would therefore be borne directly by the owners

Further analyses of the outcomes of repair and seismic retrofit of damaged buildings will reveal the effectiveness of the policy change in terms of achieving seismic resilience for Christchurch.

5 PRELIMINARY LESSONS LEARNT

5.1 Gravity-secondary elements detailing

Prior to the 1995 New Zealand Concrete Standards (NZS3101 [20]), the gravity-only and lateral load systems of structures were often designed and detailed independently. While the gravity/secondary elements were not considered as lateral-load resisting components, they deform along with the relatively flexible lateral-load system. Such "displacement compatibility" requirement may damage components of the gravity systems if they are not detailed adequately for the lateral deformation demands.

An example of this is in the fourteen storeys building shown in Figure 10, in which the exterior precast concrete frames were designed mainly for gravity. The global displacement, as the ductile walls deformed, imposed a higher-than-expected ductility demand on the limited ductility perimeter frames, thereby resulting in plastic hinging as shown in Figure 10.

The car-park building shown in Figure 19a consists of three storey RC gravity frames in conjunction with an eccentric-braced frame (EBF) system. The EBF shear links yielded at many locations, suggesting significant deformation of the overall structure. On the top storey, several columns were

heavily damaged with a bi-directional shear-flexural failure as shown Figure 19b. As steel K-braces were used in both directions, these columns were designed pre-dominantly for gravity loading. Damage to these columns indicates that the type and level of drift demand was underestimated.



Figure 19: Gravity column failure at car-park structure and similar observation in the laboratory testing by Boys et al. [21]. (Photo credit: top and left: Stefano Pampanin; right: Alistair Boys).

As demonstrated in a laboratory test on "modern" (post-1970s) columns with pre-1995 detailing, as shown in Figure 19c, the drift/deformation capacities of these columns are very limited [21]. Extensive shear damage due to deformation demand is expected in particular when subjected to bi-directional testing.

Interestingly, only the top floor columns of the building shown in Figure 19a-b were damaged. The damaged columns were possibly damaged with the displacement demand amplification due to the torsional deformation of the top floor. The top floor was intended to be torsionally-restrained by a ramp to the top floor, which was left unfinished. Further investigation of this structure is required to clearly understand the cause of the column failure.

5.2 Sign of brittle failure mode

Many pre-1970s low-rise and mid-rise RC buildings performed relatively well in this earthquake. As discussed in Sections 3.1 and 3.2, the masonry infill walls, while not part of the engineered lateral load system, contributed to the global stiffness in the early stages of the strong ground motion.

This is consistent with the observation of infill panels cracking and damage, such as those of St. Elmo Courts (Figure 7). However, the effects of interaction between infill walls and RC frames are well-known to be two-fold and controversial [22]. Masonry infill walls can increase the stiffness and strength of the bare frames structure, allowing it to survive an earthquake with an almost elastic behaviour. However, as with any brittle system, a minor exceedance of the infill's capacity can lead to sudden and catastrophic failure.

As these infill panels are generally brittle, reliance on the (damaged) infill walls for lateral strength of these pre-1970s building is considered inappropriate and non-conservative. Further cycles of strong ground motions (aftershocks or a subsequent large event) can cause severe damage in the infill walls, with sudden reduction of stiffness at a storey level, thus easily resulting in a soft-storey mechanism and/or pronounced inelastic torsional effects. Similarly shear-sliding failure and the subsequent interaction with the bare frames can cause brittle column shear failure.

Signs of incipient, but not yet fully developed, brittle failure modes within the pre-1970s RC frame buildings are also observed in many of these buildings. Figure 5 and Figure 20 present some examples of lightly reinforced beam-column joints and columns with signs of inelastic deformation during this event. Residual joint crack widths of 2-3 mm were measured in some buildings, indicating frame distortion of up to 0.75 to 1.0% inter-storey drift (based on the authors' laboratory test experience).



Figure 20: Visible residual cracks in the infills, column and joints (Photo credit: Weng Y. Kam).

Without a doubt, a ground motion of different characteristics (in terms of duration and frequency-energy contents rather than the peak ground acceleration) may impose higher deformation demands on these buildings. In absence of ductile failure modes, these pre-1970s RC buildings could fail in a catastrophic manner in similar or larger events, consistent with the experience from overseas earthquakes on RC buildings with similar forms of construction and design practice as found in New Zealand [23].

5.3 Beam elongation and precast flooring in modern RC buildings

Several instances of wide cracks on precast concrete flooring units (T-beams) and along the transverse beam and floorings unit suggest that beam elongation up to 5-10 mm could have occurred in these buildings. Figure 21 shows some examples of this type of damage in modern high-rise building. Slab mesh fracture was observed in floor topping close to the beam plastic hinges. In some instances, slab-wall connections fractured along one whole side of a building.

Wide cracks were observed in beams suggesting yielding of reinforcing bars and formation of plastic hinges. Residual cracks in these beams indicate that the elongation of the beam. The overall elongation of the beams can be estimated by adding the residual crack widths in the beams. In several cases, it was more than 5 mm elongation at a beam-column joint. As expected, the elongation of beams in a seismic frame created tension in the connection between the precast floors to transverse beams. This mechanism resulted in wide cracks in the precast floor transverse to beam interfaces, as seen in Figure 21b.



Figure 21: a) Cracks along the T-beam flooring unit perpendicular to the hinging beam; b) Beam-elongation induced slab mesh fracture. (Photo credit: John Hare of Holmes Consulting).

5.4 Precast hollowcore floor performance

Hollowcore flooring systems with pre-2003 construction detailing were shown to be vulnerable to the incompatibilities between the floor system and intrinsic deformations of the lateral resisting frames (e.g. torsional and beam elongation effects). Following a comprehensive experimental and analytical investigation as well as an industry survey, preliminary guidelines for design and assessment of hollowcore systems have been prepared [24].

A few instances of such incipient hollowcore failure mechanisms are shown in Figure 22: a) spalling of the concrete at the edge seating support, b) shear cracking of the hollowcore unit and c) damage in the negative moment region with signs of topping delamination. This damage suggests a number of possible failure mechanisms, including the loss of end seating support and flexural-shear failure of the hollowcore.

According to recent experimental results [25], depending on the geometry of the lateral resisting frame, very low ranges of lateral drift (0.25-1.0%) would be sufficient to initiate such failure modes. The fact that this type of mechanism was not widely reported in the city and that these specific instances (in Figure 22) were collected in the same car park building previously described in Section 5.1, could be seen as a further confirmation of the generally low level of displacement demand experienced by most of the structures during the 4 September event.



Figure 22: *Precast hollowcore floor and seating damages. (Photo credit: Stefano Pampanin).*

5.5 Non-structural and content damages

Although noticeable structural damage was limited to a small proportion of RC buildings, damage to non-structural components and contents was apparent in many buildings. The non-structural components damaged in RC buildings were: stairs, internal walls, wall linings, ceilings, windows, and facades.

As most RC buildings hosted commercial entities and offices, damage to contents was non trivial. In several cases, the commercial facilities had to incur significant downtime due mainly to the non-structural and contents damage despite sustaining little structural damage; the length of downtime varied depending on the building usage. As an extreme case, the main library of the University of Canterbury (Figure 23b) has been closed until the next academic semester, due to severe damage to books and shelves.

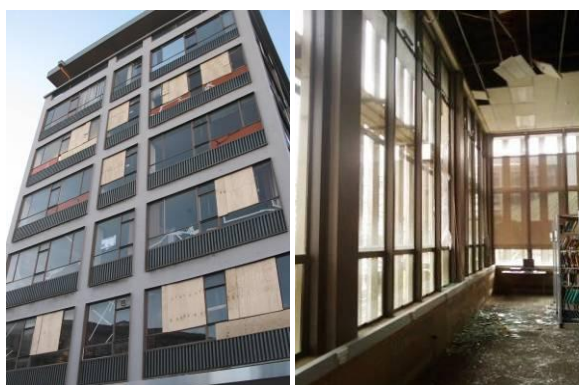


Figure 23: *Non-structural glass glazing and ceiling damages on RC frames built in the 1950s and 1970s. (Photo credit: left: Weng Y. Kam and right: NZSEE Clearinghouse).*

The future challenge of modern buildings especially for those buildings in which the owners and the functions demand higher seismic performance is to minimise and mitigate non-structural and content damage. Non-structural and content

damage in structures of all types are discussed in greater details in another paper within this issue.

5.6 Emergency stairways access and damage

Another significant concern regarding the seismic performance of modern high-rise RC buildings relates to the non-structural damage in emergency stairways, and the resulting loss of emergency egress.

Many stairways were designed to be free-hanging and to slide on the lower storey (in order to remove them from the lateral system load path). However, due to construction debris and other maintenance issues, some staircases were grouted at these seismic gaps. Consequently the staircases carried some seismic loads (as a diagonal strut) and interacted with the lateral systems during the earthquake. Floor lining damage and 10-50 mm differential levels of the staircase and the landing have been reported. Figure 24 shows an example of such damage.

Such damage is of particular concern as fire, health and safety regulations will restrict the use of a building, despite relatively unscathed main structure, if safe egress can not be ensured.

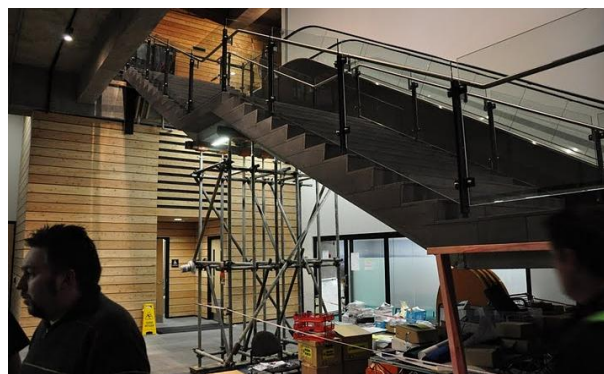


Figure 24: *Stairways interaction with lateral systems resulting in non-structural damage. (Photo credit: Fred M. Turner).*

6 FINAL REMARKS

The 4 September 2010 Darfield (Canterbury) earthquake is the first large earthquake close to a large New Zealand urban center since the 1931 Hawke's Bay earthquake. The 1931 earthquake accelerated the introduction of seismic building codes in New Zealand [23]. The impact and consequence of the Darfield earthquake are yet to be fully understood.

Seismic resistant design and good construction practice of RC buildings was evident. The seismic performance of engineered (modern and retrofitted) RC buildings, given the intensity of the ground shaking (in the form of spectrum demand and duration), was as expected by the professional engineering community.

The RC buildings built prior to the current seismic code showed signs of incipient brittle failure modes such as onset of failure in masonry infill walls, column hinging and joint shear failures. The apparently positive contribution of masonry infill walls on the pre-1970s RC buildings could have been negated if the infill panels further deteriorated (e.g., if the duration of strong ground motion had been longer). Uncertainty regarding the seismic vulnerability of older RC buildings is evident from their mixed performance, ranging from very good to moderate-to-poor.

Modern RC buildings performed very well with the exception of several notable issues. The importance of detailing secondary and gravity-only elements to "follow" the main lateral resisting systems, according to the displacement

compatibility principle, has been highlighted. Evidence of beam-elongation induced damage in precast flooring units was observed. Non-structural and content damage remains an unsolved issue in terms of maintaining building occupancy, continuity and functionality. Damaged emergency stairway and egress is noted as a building health and safety issue and may affect re-occupancy in some buildings.

Several of the retrofitted RC buildings inspected performed very well. As engineers continue to revisit seismic retrofitted structures, additional insights may be revealed. However, considering that most of the non-retrofitted RC structures survived the earthquake with low-to-moderate damage, it is hard to quantitatively evaluate the margin of improved performance or resilience provided by the retrofit intervention. The newly introduced EPB repair and retrofit policy is raising concerns regarding the burden of cost for the seismic retrofitting to the required level of 67% of current codes.

The influence of the ground motion characteristics on the seismic performance of RC buildings, such as the lack of short period / high frequency shaking, the lack of attenuation due to very soft subsoil of Christchurch City and the earthquake duration, need further analyses before any more reliable conclusions can be made.

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