SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS IN THE 22 FEBRUARY CHRISTCHURCH (LYTTELTON) EARTHQUAKE

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SUMMARY

Six months after the 4 September 2010 M_w 7.1 Darfield (Canterbury) earthquake, a M_w 6.2 Christchurch (Lyttelton) aftershock struck Christchurch on the 22 February 2011. This earthquake was centred approximately 10km south-east of the Christchurch CBD at a shallow depth of 5km, resulting in intense seismic shaking within the Christchurch central business district (CBD). Unlike the 4 Sept earthquake when limited-to-moderate damage was observed in engineered reinforced concrete (RC) buildings [35], in the 22 February event a high number of RC Buildings in the Christchurch CBD (16.2 % out of 833) were severely damaged. There were 182 fatalities, 135 of which were the unfortunate consequences of the complete collapse of two mid-rise RC buildings.

This paper describes immediate observations of damage to RC buildings in the 22 February 2011 Christchurch earthquake. Some preliminary lessons are highlighted and discussed in light of the observed performance of the RC building stock. Damage statistics and typical damage patterns are presented for various configurations and lateral resisting systems. Data was collated predominantly from first-hand post-earthquake reconnaissance observations by the authors, complemented with detailed assessment of the structural drawings of critical buildings and the observed behaviour.

Overall, the 22 February 2011 M_w 6.2 Christchurch earthquake was a particularly severe test for both modern seismically-designed and existing non-ductile RC buildings. The sequence of earthquakes since the 4 Sept 2010, particularly the 22 Feb event has confirmed old lessons and brought to life new critical ones, highlighting some urgent action required to remedy structural deficiencies in both existing and "modern" buildings. Given the major social and economic impact of the earthquakes to a country with strong seismic engineering tradition, no doubt some aspects of the seismic design will be improved based on the lessons from Christchurch. The bar needs to and can be raised, starting with a strong endorsement of new damage-resisting, whilst cost-efficient, technologies as well as the strict enforcement, including financial incentives, of active policies for the seismic retrofit of existing buildings at a national scale.

1 INTRODUCTION

1.1 General

Six months after the 4 September 2010 M_w 7.1 Darfield (Canterbury) earthquake, the M_w 6.2 Christchurch (Lyttelton) earthquake struck Christchurch on the 22 February 2011. The M_w 6.2 was centred approximately 10km south-east of the Christchurch central business district (CBD) at a shallow depth of 5km, resulting in intense seismic shaking within the Christchurch CBD.

Unlike the 4 Sept earthquake event, when limited-to-moderate damage was observed in engineered reinforced concrete (RC) buildings [35], after the 22 February event about 16 % out of 833 RC buildings in the Christchurch CBD were severely damaged. Whilst there was no fatality in 4 September earthquake (also due to the time of occurrence i.e. at 4.35am), there were 182 fatalities in the 22 February earthquake (occurring at 12.51pm), 135 of which were the unfortunate consequences of the complete collapse of two mid-rise RC buildings.

This paper describes immediate observations of damage to RC buildings in the 22 February 2011 Christchurch earthquake. Some preliminary lessons are highlighted and discussed in light of the observed buildings performance. Damage statistics and typical damage patterns of various configurations and lateral resisting systems of RC construction are presented. Data was collated from predominantly first-hand post-earthquake reconnaissance observations by the authors, complemented with detailed assessment of the structural drawings of critical buildings and the observed behaviour.

1.2 RC buildings performance in the 4 September 2010 Darfield earthquake

The seismic performance of RC buildings in the 4 September 2010 M_w 7.1 Darfield earthquake has been reported and

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discussed in previous reports, published prior to the occurrence of the 22 February earthquake [23, 35, 59].

In general, RC buildings regardless of vintage performed well and as expected, given the shaking intensity of this event, as recorded in the CBD where most of the multi-storey RC buildings are located. No RC building collapsed during the 4 September earthquake. 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 low-to-mid-rise RC buildings.

As is becoming more evident in recent earthquakes overseas, even when structural damage was limited or negligible the non-structural damage including stairway-structure interaction, ceilings and partitions was the main contributor of losses and downtime for the majority of the RC buildings.

In the 4 September earthquake, the acceleration and displacement response spectral ordinates were in general comparable or lower than the New Zealand Loading Standards NZS1170.5:2004 [41] for a 500-years return period design level for most short periods (low-to mid-rise buildings).

In the long period range (T=1.5s to 3.0s), the 4 September earthquake response spectral ordinates were generally exceeding the 500-years return period design level, indicating a moderate level of sustained damage/ductility of high-rise RC buildings. Beam plastic hinging, floor slab cracking and fracture of diaphragm topping mesh were observed in several high-rise buildings. Damaged emergency stairway and egress in high rise building was noted as a potential building health and safety issue (e.g. [35]).

2 SEISMIC SHAKING INTENSITY AND RESPONSE SPECTRA

2.1 Elastic acceleration response spectra

The elastic acceleration response spectra (5%-damped) of the 22 Feb earthquake, derived from four recorded ground motions in the Christchurch CBD are shown coefficient in Figure 1. The NZS1170:5 [41] 500-years and 2,500-years design spectra for Christchurch site (Z/PGA=0.22g), distance to nearest fault, R = 20 km and soil class D (consistent with the four recording sites) are also plotted in the same figure.

It is important to note here (further discussion in the following paragraph briefly describing the evolution of code-provisions in New Zealand), that the older (1965, 1976 and 1984) code-design coefficients have to be adjusted to become equivalent elastic spectra to allow for a reasonable comparison with the more recent NZS1170:5:2004 [41] elastic design spectra. In fact, a nominal ductility of four was assumed for those older codes. In reality, based on current knowledge, it could be argued that the actual ductility achievable by those structures (capacity) is likely to be half (approximately two) for buildings designed to the 1965 standard and closer to the assumed ductility of four for buildings designed to 1976 standard.

Some key observations of the response spectra in relation to seismic performance of reinforced concrete buildings:

- The principal component of horizontal shaking is the East-West direction. This is consistent with the observed buildings damage in the Christchurch CBD, where buildings are more damaged along the East-West direction.
- The East-West components were approximately 15-30% higher in the periods ranging from 0-2.4 s, except for the period range of 0.35 s-0.6 s in which the North-South components were stronger.
- In general, the seismic shaking in the Christchurch CBD **significantly exceeded the 500-year return period design level**, typically assumed in New Zealand for the design of normal use (residential and commercial) buildings.
- The East-West components **were comparable or exceeded the 2,500-year return period design level** in the period range of 0.5 s-1.75 s (approximately 5-20 storeys RC buildings). The 2,500-year return period design level is typically used for the seismic design of post-disaster function buildings (e.g. hospitals).
- The 2,500-year design level (approximately corresponding to a probability of exceedance of 2% in 50% building life) is considered the most severe earthquake shaking (Maximum Considered Earthquake, MCE) to which a normal use building is likely to be subjected to, as assumed in the NZS1170:2005. At this level of shaking, a newly-designed building, designed to the minimum standards in the building code, has a small margin against collapse.

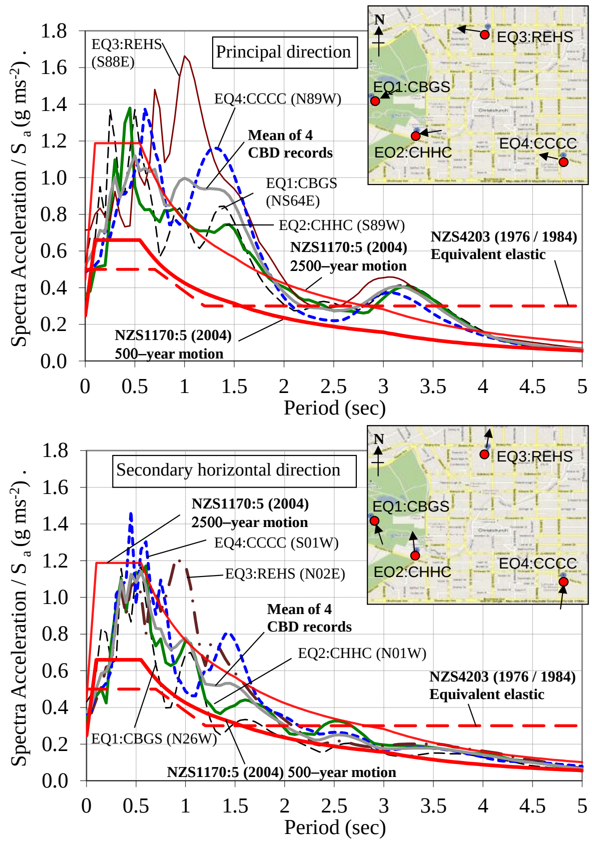

Figure 1: 22 February 2011 M_w 6.2 earthquake: Elastic horizontal acceleration response spectra (5%-damped) in the Christchurch CBD and the NZS1170.5 design spectra (red solid) for Christchurch (soil class D, R = 20 km): a) Principal horizontal direction (East-West component); b) Secondary horizontal direction (North-South component) [34].

- The amplification of spectra acceleration in the 0.5 s to 1.5 s period range and the shift of the peak spectra acceleration 'plateau' is consistent with that typically observed in ground motion records with forward directivity effects [73, 74]. The effects of such "near fault amplification" on building response are not fully understood and, more importantly, were typically not considered in the design of Christchurch buildings prior to the 22 February 2011 earthquake (caused by an "unknown" fault).

- A long period 'amplification lump' in the 2.5 s to 3.8 s period range is observed in the principal East-West component and not in the weaker North-South component. This long period amplification is likely to be a result of the basin 'slap-down' effect [6], as observed in the 4 September 2010 earthquake [13].

The equivalent vertical spectra from NZS1170.5 [42] is plotted in Figure 2 with the vertical response spectra from the **four CBD recording stations for the 22 February earthquake.** At very short period range (0.05 s < T < 0.3 s), the vertical response spectra greatly exceeded the expected NZS1170.5 vertical design spectra.

As it is difficult to determine the vertical stiffness of structures, it is hard to correlate the vertical acceleration demand to structural responses. However, in general terms, it might be expected that such a very high vertical acceleration can potentially amplify compression-loading on columns and walls, triggering axially dominated brittle failure mechanisms, induce higher gravity/seismic load on transfer elements and vertically unrestrained elements (e.g. simply-supported stair landing). It noted, however, that the high frequency content of the vertical motions resulted in the very high peak acceleration values only lasting for a *very* short duration. Further research is required to quantify the actual effects of high frequency vertical acceleration on the response of buildings subjected to a severe lateral loading.

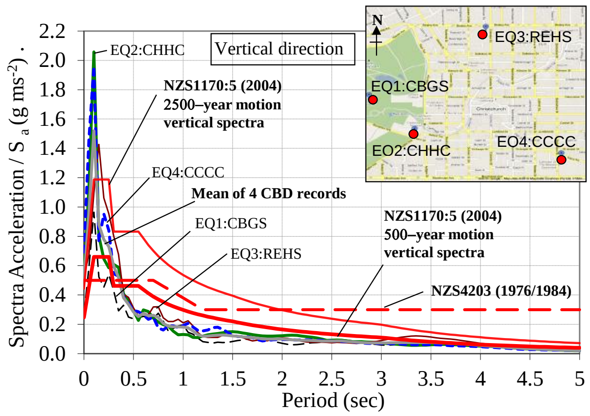

***Figure 2:** 22 February 2011 M_w 6.2 earthquake: Elastic vertical acceleration response spectra (5%-damped) in the Christchurch CBD and the NZS1170.5 design spectra (red solid) for Christchurch (soil class D, R= 20 km).*

2.2 Elastic displacement response spectra

The lateral displacement response spectra give a better representation on the seismic displacement demand and thus provide further valuable and to some extent more reliable information on the likely damage to the buildings [65]. The 5%-damped elastic pseudo-displacement response spectra for the four CBD recording stations are plotted in Figure 3.

At all period ranges, both the principal and secondary directions horizontal shaking were higher than the 500-year design pseudo-displacement spectra of NZS1170.5:2004 [41].

The elastic displacement spectra shown in Figure 3, suggests the seismic deformation demands for buildings with vibration periods (T_1= 0.8 s to 1.8 s and T_1= 2.9 s to 3.8 s) were generally very high, exceeding the NZS1170.5:2004 2500-year pseudo-displacement design spectra. This suggests that in-elastically responding RC buildings between 3 to 9 storeys and 15 to 20 storeys would have had significant displacement demands and by extension, possibly significant damage.

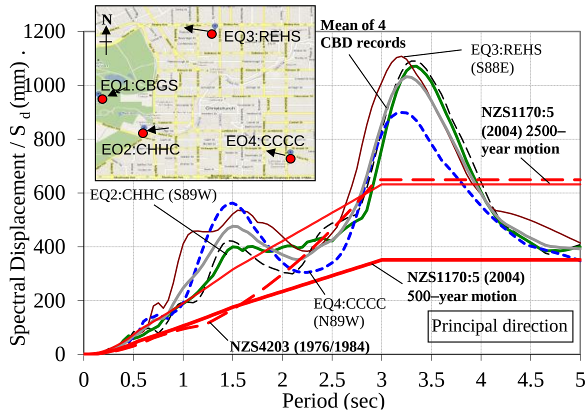

***Figure 3:** 22 February 2011 M_w 6.2 earthquake: 5%-damped elastic displacement response spectra of four Christchurch CBD records and the NZS1170.5 design spectra (red solid) for Christchurch (soil class D, R = 20 km): Principal horizontal direction (generally East-West component) [34].*

2.3 Design spectra and inelastic response spectra

In a typical "force-based" seismic design in New Zealand, the elastic 5% damped spectra will be reduced by the Ductility (k_μ) and the Structural Performance (S_p) factors following the NZS1170.5 specification. In order to compare the demand with the likely design-level capacity of modern building, Figure 4 shows the "pseudo-inelastic' or design acceleration spectra generated by reducing the individual response spectrum by an inelastic reduction factor corresponding to a ductile reinforced concrete frame structure (μ= 4 and S_p= 0.7) as per (Clause 5.2.1.1) in the NZS1170:5:2004.

For comparison, the seismic loadings for "ductile" RC frames according to the 1984 and 1976 New Zealand Loading Standards (NZS 4203:1984 [49] and NZS4203:1976 [50], respectively) and the 1965 New Zealand Loading Standards (NZS1900:1965 [43]) are also plotted as red dashed lines. For the sake of comparison it is assumed that buildings designed to these older codes will achieve the full-code compliance ductility (assumed to be μ= 4). A ductility μ of 4 is assumed to be consistent with the NZS4203:1976 and the NZS1900:1965 assumptions. Detailed retrospective comparisons of New Zealand loading standards have been published by Davenport [14] and Fenwick and MacRae [21].

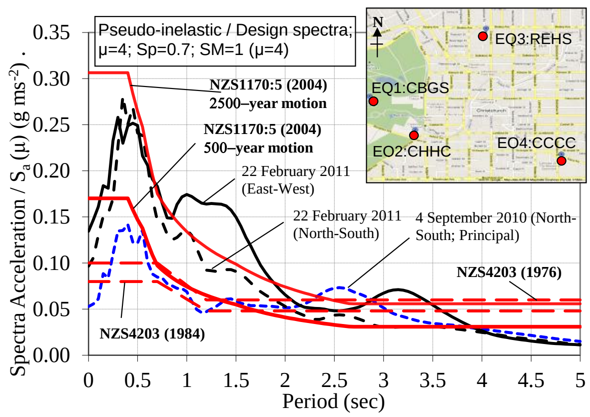

***Figure 4:** Design acceleration response spectra for the Christchurch (soil class D, R= 35 km, μ= 4 and S_p= 0.7) following the NZS1170:5:2004, NZS4203:1984 and NZS4203:1976. The pseudo-inelastic response spectra (average of 4 CBD records) for the 22 February 2011 earthquakes (both directions) and 4 September 2010 earthquake (principal direction) are also plotted.*

Effectively, Figure 4 compares the design lateral capacity or the seismic design coefficient (the lateral load capacity can be

obtained by multiplying this coefficient by the weight of the structure) for a ductile reinforced concrete frame with the implied ‘damped’ seismic action from the 22 February earthquake.

For most building periods (0.25 s < T_1 < 4.0 s), both principal and secondary pseudo-inelastic response spectra from the 22 February event exceeded the NZS1170.5:2004 500-year return-period design spectra (typical design level for normal-use). Figure 4 implies the design force (and by extension ductility and displacement) demands are exceeded by 2-3 times even for ductile reinforced concrete buildings designed to the NZS1170:5:2004 Loading Standards,

Between building periods of 0.5 s to 1.8 s and 2.8 s to 3.5 s, the seismic demands (in acceleration/forces) from the 22 February 2011 earthquakes were close to or above the NZS1170.5:2004 2,500-year return-period design spectra. In particular, these two ‘amplification lumps’ in the principal direction of the 22 February 2011 motion, indicate significant inelastic demand on structures with effective periods within these range (e.g. base isolation, flexible structures).

Interestingly, the older NZS4203 (1976 and 1984) and NZS1900 seismic coefficients are generally lower in the short periods (T_1< 0.6 s) and higher in the long periods (T_1> 1.4 s-1.6 s) when compared with the NZS1170.5 design spectra for a similarly ductile RC frame. On the other hand, it should be emphasised that while the seismic design acceleration/forces are discussed herein, the ductile detailing and other design aspects have significantly improved over time, resulting in a higher likelihood to achieve the assumed ductility (capacity to displace in the inelastic range) implied in the loading standards.

Figure 5 shows design level versus demand within an Acceleration-Displacement Response Spectrum (ADRS) domain (commonly used in seismic assessment procedures). In such domain, the building periods are plotted on radial lines. It can be observed that from an acceleration and displacement demand perspective, the 22 February event greatly exceeded the 500-year design level in most period ranges, and significantly exceeded the 2,500-year design level at several period ranges.

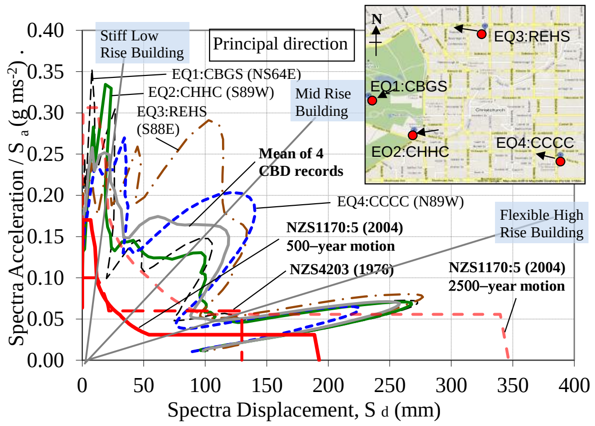

***Figure 5:** 22 February 2011 6.2 Mw earthquake: Inelastic Acceleration-Displacement Response Spectrum (ADRS) (principal horizontal direction) for the four Christchurch CBD records and the NZS1170:5 design spectra (red solid) for Christchurch (soil class D, R= 20 km, μ= 4 and S_p= 0.7).*

2.4 Remarks on seismic shaking intensity of the 22 February 2011 versus 4 September 2010 earthquakes

As observed in the comparison of pseudo-inelastic acceleration spectra in Figure 4, the ground shaking intensity, in terms of the seismic acceleration response spectra in the Christchurch CBD was about **two to three times higher** in the 22 February 2011 6.2 M_w earthquake when compared to the 4 September 2010 7.1 M_w earthquake.

In a more general contextual report, Kam and Pampanin [34] provides a more thorough discussion of the response spectra of the 22 February 2011 earthquake, in comparison with the 4 September 2010 and 26 December 2010 earthquakes.

Preliminary seismological investigation indicates the complex seismic wave interaction at the deep alluvial soils underlying Christchurch (‘basin effect’), the shallowness of the rupture and the directivity effects from the oblique-reverse fault rupture mechanism resulted in severe ground shaking within the Christchurch CBD [13, 22, 27].

Fundamentally, the occurrence of the 22 February 2011 and 4 September 2010 earthquakes and their impacts clearly confirmed the high dependency of the seismic performance of the structures to the peculiar characteristics of the ground shaking of the site (not simply limited to peak-ground acceleration or earthquake magnitude)

From the seismic design perspective, whilst the 22 February event is said to be a very rare event (in the order of 1 in 10,000 years [28]), it is apparent that a seismic design loading purely based on a uniform hazard spectra derived from a probabilistic seismic hazard model (e.g. NZS1170.5:2004) may lead to a very un-conservative and highly undesirable design outcome. Preliminary SESOC observations [72] indicate that a higher seismic design load has negligible cost impact on new buildings.

The seismic Hazard Factor ((NZS1170:5)[41] Z factor) for Christchurch and Canterbury region was elevated from 0.22 to 0.3 in May 2011, in view of the clustering effect of the seismic activity [16, 25].

A University of Canterbury Structural Group report [10] commissioned by the Canterbury Earthquakes Royal Commission recommended a dual approach to raise the bar of seismic resilience of structures: on one hand increasing the seismicity; on the other supporting the wide implementation of new technologies for damage-resistant systems, which can have comparable if not lower costs than traditional solutions .

3 REINFORCED CONCRETE BUILDING STOCK IN CHRISTCHURCH CBD

3.1 Reinforced Concrete Buildings Distribution and Types in the Christchurch CBD

Christchurch CBD is defined by the grid road network bounded by the four avenues (Deans, Bealey, Fitzgerald and Moorhouse). Christchurch CBD consists of predominantly commercial and light-industrial buildings (58%) but also contained significant number of residential buildings (42%), particularly towards the north and east edges of the CBD.

The majority (~81%) of the buildings (of all construction types) in the Christchurch CBD were of one to two storeys buildings. There were 127 buildings of at least six-storeys, with the tallest RC building being 22-storeys (86 metres).

RC frames and RC walls are the most common multi-storey construction types. Out of 175 buildings with 5- or more storeys, 51.5% are RC frame buildings, 25% are RC wall buildings, 13% are reinforced concrete masonry (RCM) and 6% are RC frame with infills. Only 9 steel structures with 5- or more storeys were observed in the CBD.

RC building construction began to flourish after the Hawke’s Bay 1931 M_w 7.9 earthquake and the associated decline of unreinforced masonry (URM) construction. Many of the mid-rise and high-rise reinforced concrete buildings in

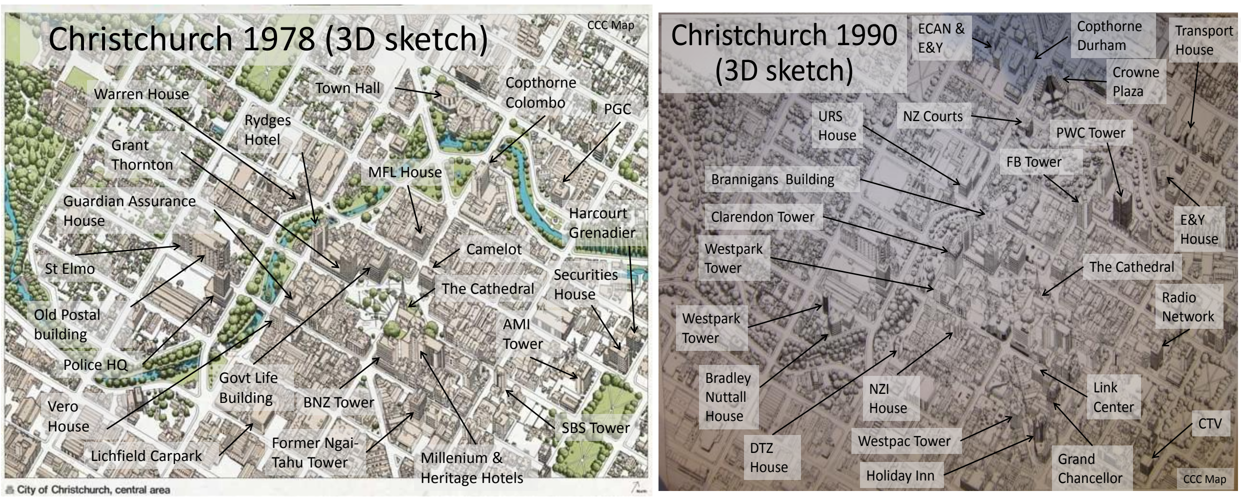

Figure 6: Notable mid- and high-rise buildings in Christchurch CBD in 1978 and 1990 [56]. Photo sketches are courtesy of CCC Library.

Christchurch were built during the construction booms in the 1960s and 1980s. Figure 6 illustrates some of the notable mid- and high-rise buildings in the Christchurch CBD in 1978 and 1990.

Buildings constructed prior to the introduction of modern seismic codes in the mid-1970s are still prevalent in the Christchurch CBD. Approximately 45% of the total CBD building stock were built prior to the 1970s. Of this, 13.8% or 188 pre-1970s buildings are 3-storeys and more, resulting in significant life safety risk in the event of collapse. Assessing and mitigating these potentially significant-collapse buildings is an internationally-recognised key priority of seismic risk mitigation.

Precast concrete floor systems began to be used for multi-storey RC buildings in New Zealand from the mid-1960s onwards. From the 1980s to present, the majority of multi-storey RC buildings use precast concrete floors or concrete composite steel deck systems. Similarly ductile precast concrete emulative (to cast-in-place approach by wet connections) frames construction was introduced in the early 1980s and soon became the most popular form of construction for RC frames.

RC shear walls, coupled-walls and dual frame-wall systems were also widely used in New Zealand from the 1970s onwards, driven by the design guidance from the research of Professors Park and Paulay at the University of Canterbury.

3.2 Reinforced Concrete Buildings Building Safety Emergency Placard / Damage Statistics

As with the 4 Sept earthquake, emergency response teams of structural engineers carried out the Building Safety Evaluation (BSE) procedure (i.e., coloured-placard tagging [53]) under the Civil Defence state of emergency authority.

While the building BSE tagging status is not a direct representative of damage, it is the best-available indicator of observed damage in a systematic format and based on a fast visual screening (exterior and interior only. Due to the rapid nature of the BSE screening for immediate risk, the tagging damage data should be interpreted with some care depending on the final purpose of the study. Further detailed damage and seismic assessment, based on structural/construction drawings and material properties, is required to establish and confirm the structural integrity of the buildings and arrive at more reliable statistics of damage.

Figure 7 and Figure 8 summarise the key statistics and findings from the processed BSE building database. The breakdown of the BSE placard statistics according to the type of building construction and year of construction is presented in Figure 8. For completeness, the statistics for all building types is also presented in Figure 8.

There are at least 3000 buildings within the Christchurch CBD (based on the 12 June 2011 CCC Building Safety Evaluation (BSE) statistics). As per 12 June 2011 (a day before the 13 June M_w 5.5 and 6.0 aftershocks), 53% of these were assessed as “Green – No restriction on use or occupancy”, 23% as “Yellow - Restricted Use” and 24% as “Red – Unsafe”.

As per 12 June 2011, 66% to 70% of “Green” and “Red” tagged buildings have had only a Level 1 rapid exterior inspection. As there is no current legislative requirement for Level 2 assessments or detailed post-earthquake seismic assessment for all the building stock (especially for green-tagged buildings), it is hard to ascertain whether the damage statistic is completely accurate. Canterbury Earthquake Recovery Authority (CERA) and CCC are currently developing requirements and technical guidelines for detailed post-earthquake seismic assessment [1].

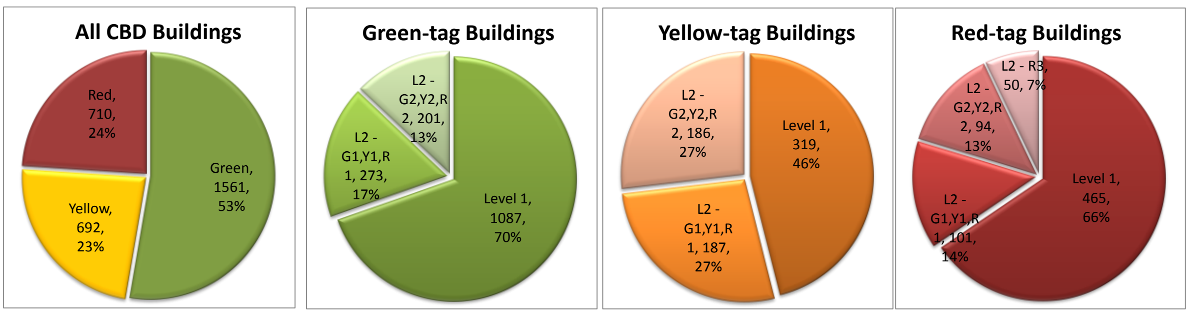

Figure 7: Distribution of buildings tagging statistics in Christchurch CBD. Building tagging is based on the CCC/Civil Defence Building Safety Evaluation procedure. (Statistics data is updated to 12 June 2011)

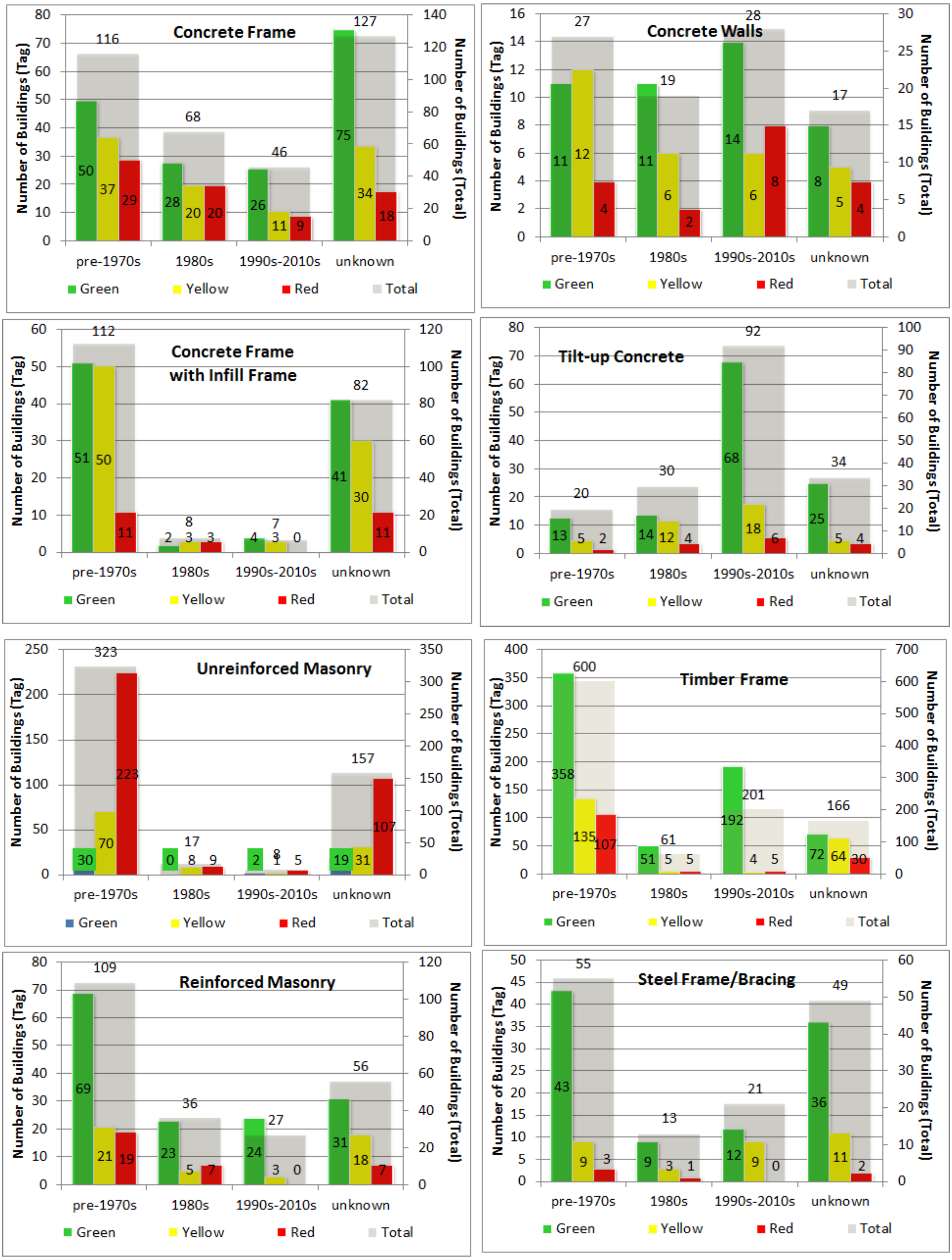

Figure 8: Distribution of Building Safety Evaluation placards of all buildings in the Christchurch CBD as per 12 June 2011 (source: CCC). The data is categorised into building construction age and the primary structural system (adapted from the CCC database, Civil Defence BSE data and authors' field inspection). The shaded bar on the secondary vertical axis shows the total number of buildings in each building construction age.

24% of all CBD buildings are Red-tagged and 23% are yellow-tagged. This represents over 1,400 buildings out of approximately 3,000 building stock in the CBD (in the available record). In a previous CERA estimation, up to 1,300 buildings may be demolished [31].

Table 1 summarises the distribution of BSE tagging of the 833 inspected RC buildings within the Christchurch CBD area as of 12 June 2011. The placard distribution for the 717 RC buildings inspected within the Christchurch City Council (CCC) boundary after the 4 September event is shown in reference [35].

Table 1. Distribution of Building Safety Evaluation placards of all RC buildings in the Christchurch CBD as per 12 June 2011 (source: CCC).

Types of Constructions	NZSEE Building Safety Evaluation Tagging		
	Green	Yellow	Red
Reinforced Concrete (RC) Frames	179 (50.1%)	102 (28.6%)	76 (21.3%)
RC Shear Wall	44 (48.4%)	29 (31.9%)	18 (19.8%)
RC Frames With Masonry Infill	98 (46.9%)	86 (41.1%)	25 (12%)
Tilt Up Concrete	120 (68.2%)	40 (22.7%)	16 (9.1%)

Evidently, the statistics indicate a significantly higher number of Yellow and Red-tagged buildings in the 22 February earthquake, when compared with the 4 September earthquake where nearly 90% of all RC buildings inspected were given a Green tag [35].

There is a consistent trend of higher observed damage or proportion of yellow/red tagged buildings constructed prior to the 1970s, for all construction types. More than 54% of the pre-1970s RC buildings (RC frames, walls, infilled frames or tilt-up walls) were tagged as Yellow or Red. In comparison, about 44% of the post-1970s RC buildings were tagged as Yellow or Red. While the percentage of severely damaged 1970s RC buildings was expected, the higher-than-expected percentage of post-1970s RC buildings damaged (or Yellow and Red-tagged buildings) was somewhat unexpected considering the improvements in the seismic provisions.

The introduction of modern seismic codes in the 1970s also led to the significant decline of reinforced concrete infill frames buildings. Unreinforced masonry (URM) construction was in general ceased after the 1931 Hawkes Bay earthquake.

4 GENERAL PERFORMANCE OF PRE-1970 RC BUILDINGS BUILT

In the following discussion, the classification “pre-1970s” and “modern buildings” refers to buildings designed prior-to and after the 1976 “modern” seismic code NZS4203:1976 [49] respectively.

Without explicit design for lateral-force resistance, ductile detailing and capacity-design concepts, for example, buildings constructed prior to NZS4203:1976 and NZS3101 concrete codes [45, 46] provisions generally have inadequate seismic capacity and brittle failure modes.

Typical structural deficiencies of pre-1970s RC buildings are: a) Lack of confining stirrups in walls, joints and columns; b) Inadequate reinforcing and anchorage details; c) Poor material properties and use of plain reinforcing bars; d) No capacity design principles’; e) Irregular configuration.

4.1 Pre-1970s RC frames-walls buildings

The seismic vulnerability and the non-ductile behaviour of pre-1970s RC frame buildings are well documented based on past research and observation in recent earthquakes [52, 54, 61]. Based on the BSE tagging statistics, up to 57% of pre-1970s RC frame buildings were either yellow or red-tagged (see Figure 8).

The catastrophic total collapse of the Pyne Gould Corp building (1960s RC frame/wall structure) draws a significant attention to the high vulnerability of pre-1970s RC buildings. Considering the total catastrophic collapse of the Pyne Gould Corp building (see Section 6.1), the wide variability of the seismic performance of these buildings will require further studies.

The poor seismic behaviour of these buildings is as expected. In many buildings, the presence of plan and vertical eccentricity and torsional amplification intensified the seismic displacement and force demands on non-ductile RC elements.

Plan irregularity and column shear failure: Figure 9 and Figure 10 show a 4-storey 1950s RC frame-wall building with severe columns damage on the Northern frame. The building is reinforced with plain round bars. As seen in Figure 10, the columns, which failed in shear, have almost negligible transverse reinforcement.

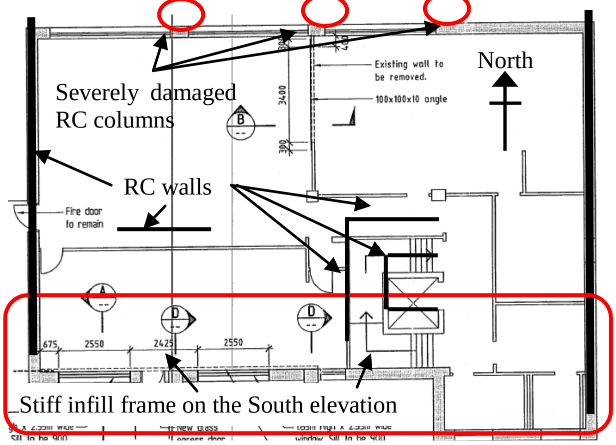

Figure 9: Plan stiffness eccentricity due to stiff infill frame and internal RC ‘non-structural’ RC walls.

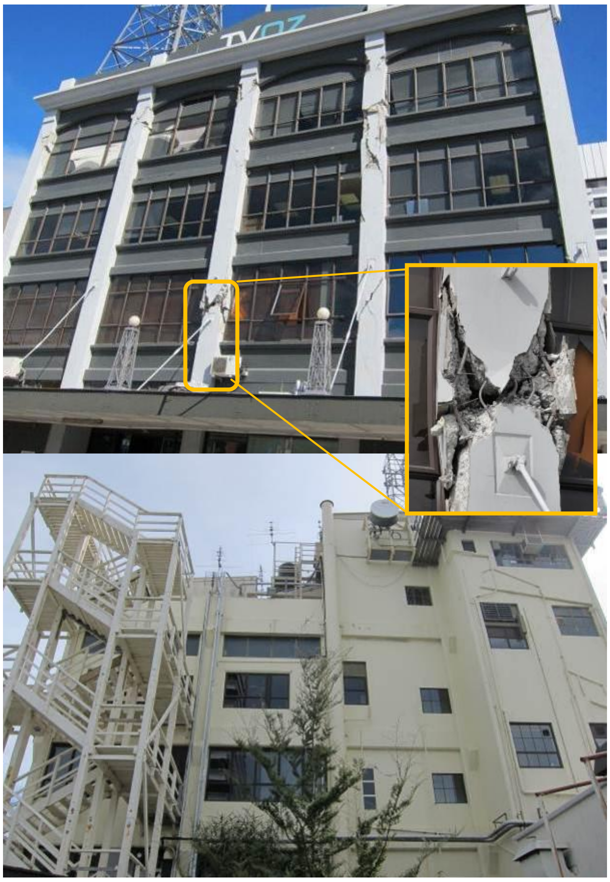

Figure 10: Severe column shear failures of the front (North) façade frame of a 4-storey RC frame-wall building.

Excessive shear demand was imposed on these columns on the Northern frame due to the plan stiffness eccentricity of the building. The plan eccentricity was a consequence of the stiff infilled RC frame and RC core walls at the South end of the building, resulting in torsional demand on the Northern frame due to East-West seismic shaking

Foundation beam, coupling wall and joint shear failure: Figure 11 and Figure 12 show a 5-storey RC frame-wall building with multiple elements failing in brittle behaviour. It comprises six one-way RC frames in the North-South direction and several coupled- and single RC walls acting predominantly in the East-West direction.

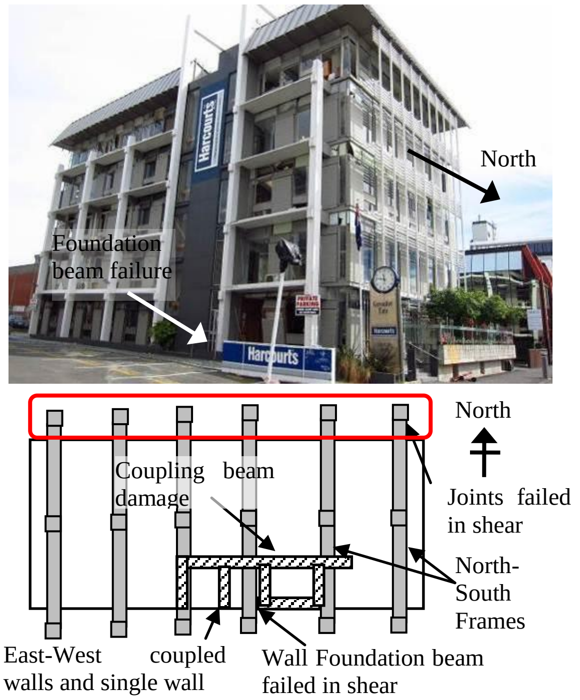

Figure 11: ***Pre-1970s RC frames-walls building with multiple elements experiencing a brittle failure mode.***

The lateral resisting system in the East-West direction appears to be severely damaged. The coupled walls in the internal grid line had severe damage on its coupling beams at the lower three storeys (Figure 12c). The coupling beams are lightly-reinforced with plain round bars. The foundation underneath the core walls around the lift-shaft appears to have failed and dropped approximately 400 mm (Figure 12b). One of the foundation ground beams was observed to have failed in shear with evidence of liquefaction observed in the vicinity of the foundation beams (Figure 12d).

It is likely that the RC frames resisted a significant portion of the lateral load in the North-South direction and torsional load from the East-West shaking. The failure of the walls system and foundation beam in the East-West direction and the vertical drop of these core walls also 'dragged' the RC frames inward, resulted in shear-failure of the beam-column joints as the frames deformed inwards. The unreinforced beam-column joints developed the highly brittle shear-wedge mechanism.

The building subsequently collapsed in an aftershock on 13 June 2011.

Short column and joint shear damage of an early 1970s building: Figure 13 shows an 8-storey 1973 building of two-way RC frames with a C-shaped core-wall structural system. Typical 457 mm square columns are reinforced with 12 distributed D28 (28 mm diameter deformed) longitudinal bars and D10 stirrups at 230 mm centres. The beam-column joints are reinforced with 1-2 stirrups. The C-shape wall is reinforced with D10 at 200 mm centres vertically and D10 at 250 mm centres horizontally.

The first floor columns on the North elevation failed in shear with the upturned spandrel beam creating a short-column effect. In both Northern and Southern elevation frames, the beam-column joints were cracked with limited spalling. No apparent damage of the shear-core wall was observed.

Figure 12: ***Pre-1970s RC frame-wall building: a) Exterior joint shear failure ; b) Approximate 400mm drop of the RC walls; c) Coupling beam failure; d) Shear failure of the foundation beam.***

By most accounts, this early 1970s RC building has performed reasonably well despite the onset of the brittle failure mode in the columns. The redundancy provided by the dual frame-wall systems ensures the building remains standing despite the onset of brittle failure of the East-West perimeter frames.

The core wall did not seem to resist a significant amount of the seismic inertial forces. Relative minor cracks were observed within the core walls. The diaphragm area (~4.5 m x 2 lengths) tied into the core wall is limited by the voids within the core and the location of the walls.

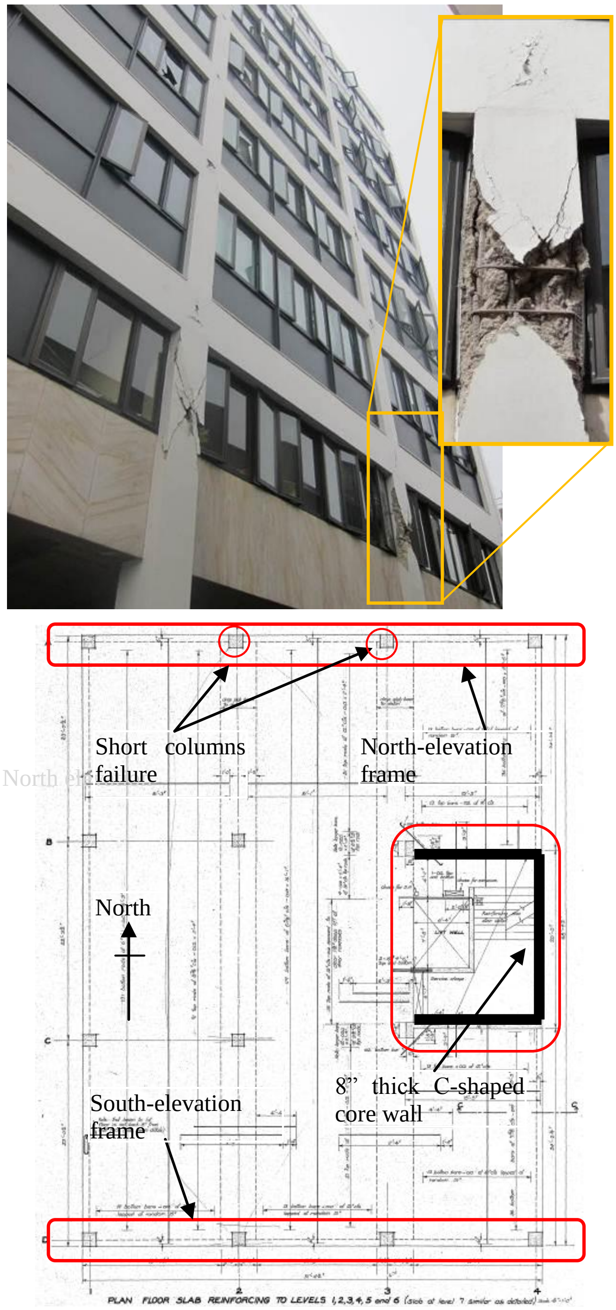

Figure 13: ***An 8-storey building with two-way RC frames system had a mix of column and beam-column joint shear failures.***

Beam lap-splice failure: Figure 14 illustrates the typical damage sustained by a 5-storey 1967 RC frame building (plan view shown in Figure 15). The building comprises six two-bay RC frames in the East-West direction and three five-bay RC frames in the North-South direction.

The tower structure on the West side, seismically isolated from the frame building, was tilting 120 mm east due to ground failure.

The majority of the 1st floor beams in the RC frames spanning in the East-West direction had beam lap-splice failures (Figure 14). From structural drawings and confirmed by site inspection, the beam’s 32 mm diameter longitudinal bars only had approximately 500 mm lap length (approximately 16d_b), with 9.5 mm diameter ties at 457 mm centres. The lap-splice failure-initiated cracking generally led to an inclined shear failure mode as the concrete shear contribution was limited.

The base (ground floor) columns are well-confined for ductility demand with 9.5 mm diameter ties at 100mm centres provided. At upper levels (2nd and 3rd floors), the East-West spanning beams had minor-to-moderate flexural cracks.

The building further deformed significantly after the 17 April 2011 5.3 M_w aftershock, with a near soft-storey collapse at the ground floor, leading to an urgent demolition order. This building illustrates how a simple critical deficiency such as beam lap-splice failure can lead to catastrophic building failure and soft-storey collapse.

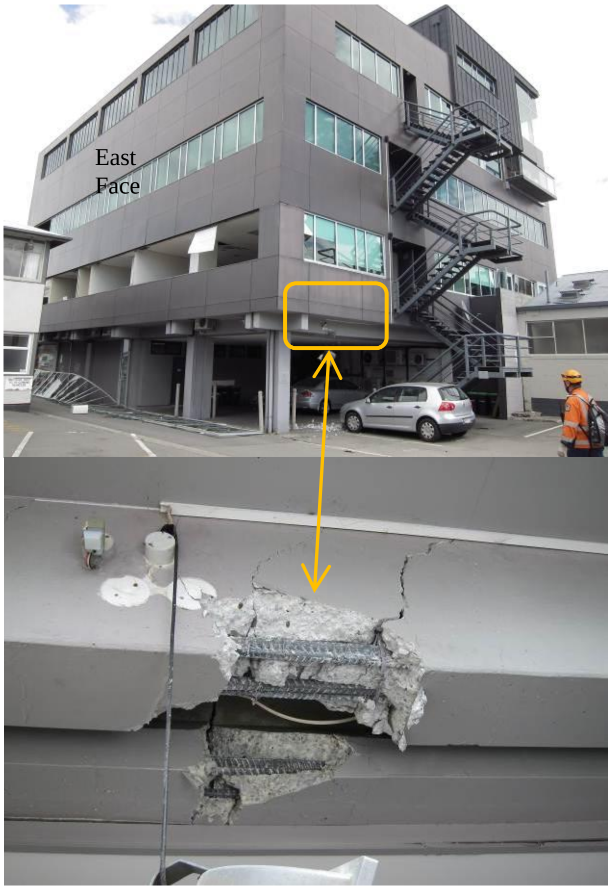

Figure 14: ***Ground floor beam lap-splice failure.***

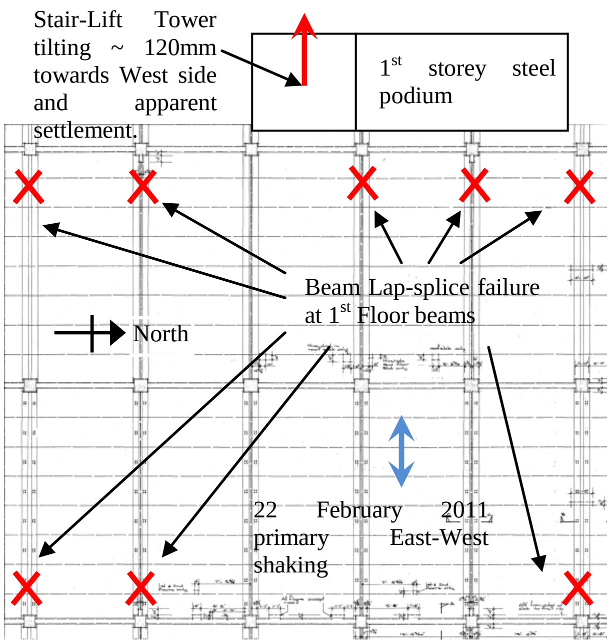

Figure 15: ***5-storey RC frame building built in 1967 with beam lap-splice failure.***

4.2 Pre-1970s RC walls

Pre-1970s RC walls are generally very lightly reinforced and are prone to shear-failure and compressive buckling failure. One or two layer of 9.5 mm (3/8”) diameter bar at 305 mm (12”) centres is the typical horizontal and vertical reinforcement provided for a typical 150 mm to 200 mm (6” to 8”) thick wall.

Prior to the NZS3101:1982, walls were not detailed for ductility with inadequate horizontal and vertical reinforcement at critical regions of the walls. In particular, the older type walls generally have no adequate reinforcement to provide confinement to the concrete and buckling restraint to the longitudinal reinforcement.

Nevertheless, some older RC walls buildings with significant structural redundancy and thicker wall sections appeared to perform satisfactorily. However, as to be discussed for the Pyne Gould Corp (PGC) building in Section 6.1, when the lightly-reinforced RC core wall is the only lateral-load resisting element and the “gravity” frames are not capable of sustaining moderate to high drift demands, the building can be susceptible to catastrophic collapse.

Expected wall shear and flexural failure: Figure 16 illustrates the typical shear-type and flexural-type failure of long lightly-reinforced RC walls in pre-1970s low-rise to mid-rise building.

Figure 16a shows the ground-floor section of RC walls of a 5-storey building with multiple cantilevered walls and coupled-walls as its lateral-load resisting system. While shear cracks have been initiated, the building has significant residual lateral strength, owing to the multiple redundancy and relatively thick walls.

Figure 16b shows one of four East-West RC walls with flexural failure at the 3rd floor of a 9-storey building built in 1964-65. The RC walls are bounded with concrete-encased steel columns. The remainder of the building structure comprises two-way steel frames (possibly moment-resisting frames) providing some lateral stability despite the failure of the shear walls. The vertical irregularity due to the one-bay setback at the 2nd floor resulted in the concentrated shear damage observed at the 3rd floor.

At the vicinity of the cracked and spalled concrete, the vertical plain reinforcement and the flange-plate of the steel columns were buckled. The inadequate bond capacity of plain-round bars after flexural-cracking resulted in one discrete crack/failure plane with significant inelastic strain demand on the exposed reinforcement.

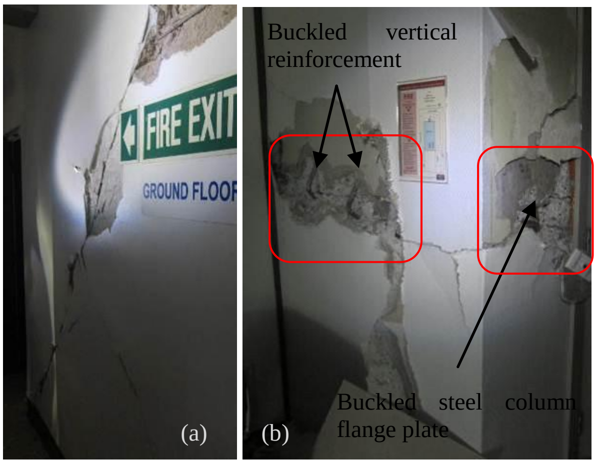

Figure 16: Typical shear and flexural failure of RC walls in buildings built prior to the 1970s.

Boundary zone crushing and bar buckling –Figure 17 shows bar buckling and crushing of wall boundary zones with light longitudinal reinforcement and confinement. The 8-storeys building designed in 1967 has four similar walls located at the four corners, all oriented in the E-W direction. The walls are roughly 4 m long and 230 mm thick, with a one-sided flange extending approximately 750 mm from the web at one end. This flange is terminated at the ground floor level and crushing is observed in all 4 walls at the top of the basement level immediately below the termination of the flange.

Bar buckling was observed on the opposite end of the wall where concrete spalling exposed the wide spacing of transverse reinforcement. Large displacement demands due to crushing at the wall base resulted in severe damage to the slab adjacent to the damaged wall in most of the upper stories and shear failure of the coupling beam at the 7th level.

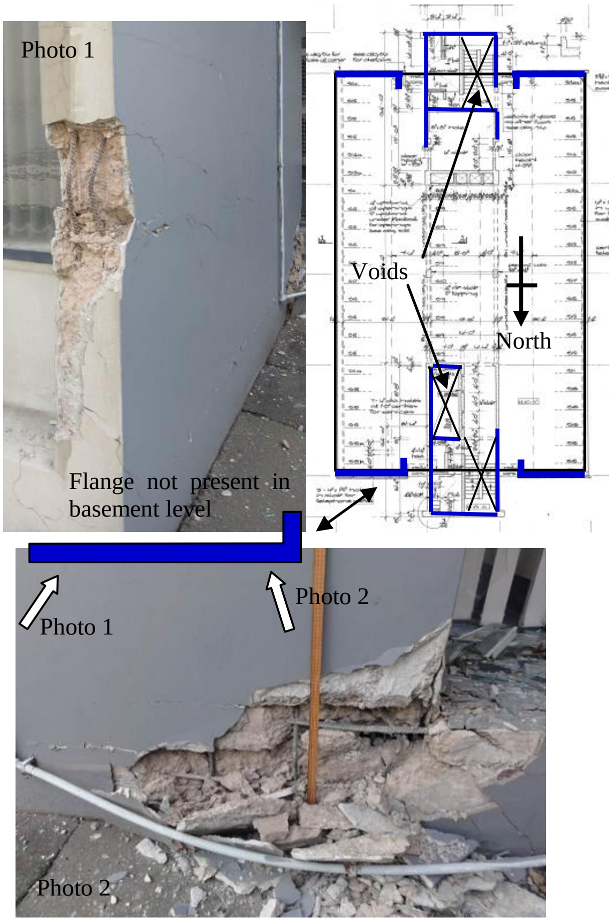

Figure 17: Wall boundary compression zone crushing and buckling failure.

Compression zone failure: Figure 17 above and Figure 18 below illustrate typical compression-zone failure of RC walls with irregular section shape. As these pre-1970s walls were lightly reinforced with almost no cross ties or confining reinforcement at critical compression section, the compression zone concrete cracked and spalled under a low level of shaking. The subsequent seismic loading cycles thus led to the buckling and/or fracture of the wall longitudinal reinforcing.

Figure 18: ***Compression zone failure of pre-1970s RC walls. Photo is taken from the ground floor wall of an early 1960s 9-storey RC building.***

Coupling-beams shear failure: RC coupled-walls were a developing ductile seismic structural form in the 1960s with various different detailing practices used to transfer the significant shear across the coupling spandrel beams. Figure 19 shows a 9-storey office building designed and built circa 1965, with significant damage to its RC coupled-wall elements.

The RC core walls (coupled walls and C-shaped walls) on the Southern elevation provide the main lateral-load resisting system, with a two-bay gravity steel frame on the Northern elevation spanning in the East-West direction. The torsional eccentricity is resisted by the coupled-walls in the North-South direction (Figure 19b-d). Despite its vintage, the coupling beams have diagonal and horizontal deformed reinforcement. No confining vertical ties however are provided in the coupling elements, leading to substantial concrete spalling after shear failure.

The coupling beams of the main lateral structural elements in the East-West direction were severely damaged at the 3^{rd} and 4^{th} floors (see Figure 19e). At the 1^{st} to 2^{nd} floors and 5^{th} to 6^{th} floors, the coupling beams damage was less severe. The vertical damage distribution indicates a strong contribution from the second mode of vibration for the building.

The staircases, which were within the confined RC core walls, were also severely damaged at their supports, particularly at the upper floors (beyond the 4^{th} floor). The stairs were supported on three 'pinned' connections with no allowance for movement. The bottom connection, consisting of a steel fixing bolted into in-situ concrete (with an apparent compressible material) was severely damaged (see Figure 62c).

Lack of load path and adequate connection between diaphragm and wall:

In several buildings, the lack of damage to some RC walls despite the apparent deformation demand on the remainder of the buildings suggests that the load path from, and connection to the floor diaphragm to the walls was poor and limited.

The 8-storey RC wall building shown in Figure 17 is an example of this. While four of the L-shaped RC walls in the East-West direction were damaged, the internal RC walls (also spanning in the East-West direction) shows a limited level of distress. The presence of voids (from services, lift and staircase penetrations) and limited diaphragm ties into the walls means limited inertia forces were transferred into these walls, despite being 'stiffer' than the L-shaped walls.

As will be discussed in Sections 5.2 and 6, the poor load path between diaphragm and wall is not limited to pre-1970s walls. The lack of integral and robust diaphragm-to-walls load paths, combined with several other factors can be catastrophic, and may have contributed to the collapse of the CTV building.

Figure 19: ***a) Seven-storey 1960s coupled-RC walls building with significant damage on the coupling beams. b) Shear walls damaged at East elevation; c-d) Coupled-wall damage at the West elevation; e) Damaged coupling beam at the internal coupled-walls (East-West direction).***

4.3 Reinforced concrete frames with infills

RC frames with masonry infill buildings can be a particularly vulnerably class of buildings, evident from the experience of overseas earthquakes. These buildings are also relatively common in New Zealand from the early 1920s to the mid-1960s, owing to the masonry infill perceived function as acoustic and fire boundaries. Therefore, the masonry infill panels along the building length usually have no openings, while the building frontage and rear elevation infill walls will generally have extensive window penetrations.

Masonry infill walls prior to the 1950s were generally unreinforced masonry clay bricks, with no seismic separation provided between the frames and the infill bricks. From the

mid-1960s, seismic gaps between the infill walls and frames were typically used [44]. The choice of infill masonry also gradually switched from unreinforced clay bricks to lightly reinforced concrete block masonry.

The seismic behaviour of moment-resisting frames with full or partial height masonry infill is very complex. If the walls are not separated from the frames, the infilled frames can behave almost like a shear wall (e.g. Figure 10) up to the premature brittle failure of the infill material. From there onwards brittle mechanisms can develop both at local (captive or short columns e.g. Figure 20) or global level (soft-storey).

Few cases of severe damage of infill frames were observed in Christchurch after the 22 Feb earthquake. Notably, one three-storey RC frame building with masonry infill building collapsed after the 13 June 2011 M_w 6.2 aftershock (Figure 20). The building in Figure 20 had localised damage such as short-column shear failure due to partial height infills and joint/column shear cracking after the 22 February event. However, with subsequent aftershocks and the cumulative strength degradation of the masonry infill walls and RC frames, the central portion of the building collapsed in the 13 June aftershock.

***Figure 20:** Reinforced concrete frame buildings collapse/damage patterns: Pre-1930s three-storey RC frame with masonry infill a) Survived the 22 February 2011 Mw 6.2 earthquake; b) Collapsed after the 13th June Mw 6.0 aftershock; insert: short column failure.*

RC frames with masonry infill walls, both unreinforced and reinforced, are generally very stiff, with the participation of infill walls can provide a lateral over-strength capacity as high as 1.5 to 2.5 times that of bare RC frame (e.g. [2]).

However, the effects of interaction between infill walls and RC frames can be both positive and detrimental. Masonry infill walls can increase the stiffness and strength of the bare frame structure, allowing it to survive a certain level of earthquake shaking with an almost elastic behaviour. As observed for the building in Figure 20, further cycles of strong aftershocks can cause severe damage in the infill walls, leading to sudden reduction of stiffness at a storey level, thus easily resulting in a soft-storey mechanism and/or pronounced inelastic torsional effects.

Figure 21 shows the flexural-shear failure of a RC masonry infill wall, which acts as both infill wall and lateral-load resisting element. The building is a two-storey rectangular shape building with RC frames in the transverse direction and RC infilled-frames in the longitudinal (East-West) direction. As seen on Figure 21, the infill wall is heavily reinforced vertically for flexure but is lightly reinforced for shear.

***Figure 21:** Flexural-shear damage of a RC masonry infill wall within two-storey RC frames.*

4.4 Reinforced Concrete Masonry (RCM)

Reinforced Concrete Masonry (RCM) is a construction material/technique that was introduced in the early 1950s and popularized in the 1960s. In particular, Christchurch pioneered the use of RCM walls as seismic resisting system for mid-rise buildings in New Zealand.

With the introduction of the New Zealand masonry code in the 1960s [44], the material quality and masonry workmanship were perceived to have significantly improved. In Christchurch, mid-rise residential buildings up to 6-storey were built using RC block masonry. Figure 22 shows some typical detailing of RCM lateral and gravity load-bearing walls used in the 1960s.

Typically observed deficiencies of RCM buildings are: a) Un-grouted cell with vertical reinforcement, b) Poor anchorage of reinforcement and foundation/bond beams, c) Lack of or inadequate horizontal (shear) reinforcement, and d) poor concrete block material.

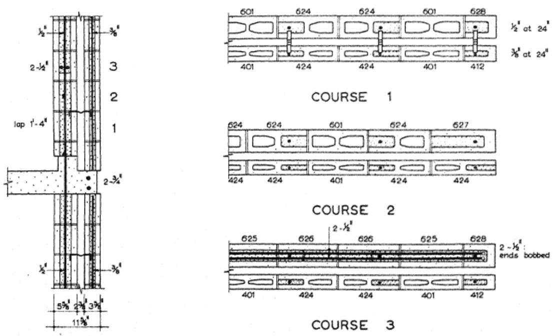

***Figure 22:** Typical detailing of RCM lateral and gravity load-bearing walls used in New Zealand in the 1960s (taken from [33]). The external veneer can be unreinforced or reinforced with no grouting.*

Figure 23 shows a 2-storey RCM residential building that suffered soft-storey collapse. Pull-out failure of the plain round reinforcement lap-connection at the base of the wall was likely to contribute to the collapse. An inspection of the lap-connection (Figure 23b-c) indicates a limited starter-bar development length (approximately 30-35 bar diameter) was

provided (for the plain bar reinforcing). The construction quality is also generally poor, with relatively porous grout material and evidence of rusting of the longitudinal bars.

Figure 23:** **a) Soft-storey collapse of a 2-storey residential RCM building. (b-c) Anchorage pull-out failure of the lapped vertical reinforcement.

Figure 24 shows one of several mid-rise RCM buildings in Christchurch. Extensive shear damage of the 1st floor transverse (East-West) walls was observed (Figure 24b and d).

The building's external wall has two layers of RCM blocks, with the grouted vertical reinforcing (shown in Figure 22 according to [33]). At the Northern side panel (Figure 24b), the failure plane was through both layers of RCM blocks. Some of the vertical reinforcement appeared to be inadequately grouted in the cells of the concrete blocks.

The interior walls are typically single-layer RCM walls. The concrete blocks were heavily damaged along the shear failure plane at the 1st floor (Figure 24d). Few grouted cores were observed.

Crushing and compressive failure of the RCM blocks was observed at the corner walls (e.g. Figure 24c). The poor grouting of the reinforcement, particularly at the outer veneer was evident. The damage was more extensive in the outer layer bricks.

The primary deficiencies in the RCM wall systems observed are generally related to poor construction quality and masonry workmanship in specific buildings. Significant development in RCM design [63] and improvement in the construction standard [51] since the 1980s would have rectified many of these deficiencies. Nevertheless, since the 1980s, RCM construction has become less commonly used for mid-rise buildings due to the lack of confidence in the material and economic reasons.

The damage of RCM walls for single-household residential dwellings, typically single-storey and not designed to the NZS4230 standards [51], have also been observed. These buildings are generally not-engineered and are built to the NZS4229 specification.

Figure 24:** **Typical shear and compressive failure of reinforced concrete masonry walls construction.

4.5 Heritage pre-1940s Reinforced Concrete buildings

In the Christchurch City Council (CCC)'s City Plan, 29 RC buildings are listed as Heritage Buildings [30], fourteen of which are between four to six storeys. Twenty-five of them were built prior to 1942.

The seismic performance of these early pre-1940s RC buildings varied significantly, depending on the building typology, redundancy within the structural system, governing inelastic mechanism and the presence of past seismic strengthening.

Figure 25 illustrates two examples of heritage pre-1940s RC buildings that survived the Canterbury earthquakes reasonably well (from preliminary exterior and internal inspections).

Figure 25a is the four-storey Victoria Mansion, a predominantly RC walls building built in 1935. It consists of multiple RC columns, RC walls and unreinforced masonry (URM) infill walls, resulting in a reasonably stiff and robust structural system with a high degree of redundancy. Preliminary inspections indicate the building suffered minor structural damage, consisting of minor shear cracks on the RC wall and column elements and separation/splitting cracks of the URM walls.

Figure 25b shows the 1926 National Bank (Isaac House) building. It comprises a two-way RC frames structure with multiple masonry infill walls on the perimeter and internal partitions. It is not known at the time of writing whether the building has been seismically-strengthened, but it is likely to have been strengthened to a degree. The regular distribution of reasonably robust RC lateral systems also helps the seismic performance of the building.

Figure 25: ***a) 4-storey Victoria Mansion (1935), with RC walls or possibly RC frame with infill walls, had limited structural damage; (b) 4-storey RC frame/wall National Bank / Isaac House (1926) showed limited cracking and damage from preliminary inspection.***

Figure 26 and Figure 27 show two examples of older 1940s RC buildings which suffered significant damage to the extent of being demolished.

St Elmo Court (Figure 26) is an 8-storey RC frame building with an internal core wall with limited capacity. The exterior façade consists of two layers of URM infill walls with a cavity gap. In the 4 September 2010 earthquake, many of the large panels of URM walls cracked and one ground floor column had diagonal shear cracking [35].

After the 22 February 2011 aftershock, many of the exterior URM walls were further damaged. Several of the interior columns at the ground floor had limited diagonal shear cracks. However, the building was considered to be a soft-storey collapse risk as the URM walls failure within a floor can result in a high stiffness irregularity.

Prior to the 22 February earthquake, conceptual seismic retrofit solutions using post-tensioned precast concrete or timber walls were considered for the damaged St Elmo building. However, the damage and uncertainty after the February event made the repair and retrofit options not viable and uneconomical.

The building was amongst the first to be de-constructed in the Christchurch CBD, due to its proximity to the main arterial traffic to the Civil Defence Headquarters, Police Headquarters and CCC Building.

Figure 26: ***The 8-storey St Elmo Courts (1935) suffered heavy masonry infill walls damage.***

Figure 27: ***The 6-storey Kenton Chambers (1929) with perimeter URM walls and interior RC frames.***

The 6-storey Kenton Chambers (Figure 27) built in 1929 comprises perimeter URM load-bearing façade walls of three brick thicknesses and interior RC frames. It has cast-in-situ RC floor on a grid of RC beams supported on RC column (Figure 27c). Several interior ground floor columns experienced flexural failures with buckled longitudinal smooth bars observed. The Northern face perimeter walls (along the East-West direction) were heavily damaged, with partial collapse of two of the six piers. The Eastern face URM wall appeared to have little damage.

5 GENERAL PERFORMANCE OF ‘MODERN’ POST-1976 RC BUILDINGS

In the following discussion, “modern buildings” refer to RC buildings designed after the 1976 “modern” seismic loading standard NZS4203:1976 [49] (with capacity design principles

introduced) and more specifically, after the introduction of the ductile detailing and implementation of capacity design for RC structures in the 1982 NZS3101 [46].

It should be noted that since early 1980s to the present, precast concrete construction, in particular in its emulative of cast-in-place approach, is used in New Zealand for most RC frames (Figure 28a-b), all RC floors and to some extent RC walls [60]. By the means of capacity design and proper connection detailing of the precast concrete elements, both cast-in-situ monolithic and precast concrete monolithic-emulation systems are expected to perform similarly under earthquake shaking [48, 60].

5.1 Modern (Post-1970s) RC Frame buildings

A construction boom in the 1980s led to a large number of mid-to-high rise RC buildings in the Christchurch CBD, for which precast concrete ductile perimeter frame systems were widely used. Some of these high rise buildings were previously reported to be damaged during the 4 September 2010 Darfield earthquake [23, 35].

Ductile beam-hinging behaviour in cast-in-situ and precast cast-in-place emulation RC frames: Many of the modern RC moment-resisting frame buildings, generally performed well and exhibited moderate-to-severe ductile beam end hinging mechanisms commensurate with the seismic excitation (e.g. Figure 28c-d). Column or beam-column joint distress/damage was not observed in most of the modern RC frame buildings inspected by the authors.

Figure 28 shows the typical beam end plastic hinging damage observed in a RC perimeter frames high-rise building. As with many high-rise RC buildings, the building's perimeter frames provide the main lateral-load resisting capacity while the more flexible interior frames are intended to carry mainly gravity loading. As observed in Figure 28, the precast concrete frames with wet connection outside the plastic-hinge zone behaved very well, with beam-hinging at the desirable locations.

It should be noted that a number of these buildings had minor to moderate levels of damage in the 4 Sept 2010 earthquake [35]. However, the building damage was typically as expected from a moderately ductile response of the RC frames in the 4 September earthquake.

It is noteworthy that some of the mid- to high-rise RC frame buildings have been considered uneconomical to be repaired, even though they have exhibited a good ductile behaviour in a severe earthquake, consistent with the design expectations according to the current seismic code (e.g. NZS3101:2006 [48]). Moving forward, the financial risk and damage acceptance of ductile RC systems may require further consideration.

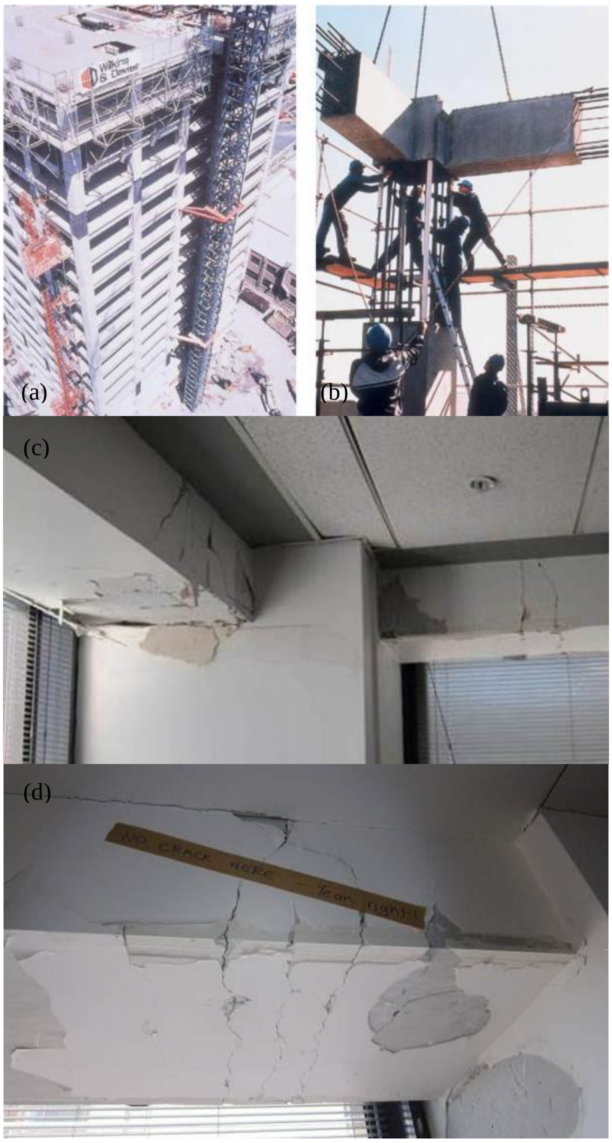

Figure 28: ***Post-1970s RC moment-resisting frame buildings collapse/damage patterns: a-b) 22-storeys precast concrete frame building under construction; c) Two-way plastic hinging on 5th floor of a 22-storeys office tower; d) Bean plastic hinge mechanism occurred in both the 4 September and 22 February earthquakes.***

Beam-elongation and precast flooring unit failure: Figure 29 and Figure 30 illustrate an extreme example in which extensive floor diaphragm damage with near loss of precast flooring unit supports occurred due to the beam elongation effect.

Displacement-incompatibility of lateral load resisting systems and the "gravity" elements such as precast floor, gravity elements and transfer beams have been recognized as a critical structural weakness in recent research [47]. In particular, the adverse elongation effect from ductile plastic behaviour of lateral system (i.e. reinforced concrete frames) on the structural integrity of the diaphragm of the precast flooring elements is well documented [20, 39].

The building shown in Figure 29 and Figure 30 is a 17-storey building with ductile RC perimeter frames, internal gravity frames and flange-hung supported precast double-tee flooring. 60mm topping with cold-drawn wire mesh reinforcement was used. The perimeter frames have typical 500x850 mm deep precast beams with 600mm square and 800 mm square columns. The beam spans are typically 2.9 m in the East-West direction and 5.8 m to 6.5 m in the North-South direction.

A ductile beam hinging mechanism in the North-South perimeter frames was observed (and repaired) after the 4 September 2010 earthquake. In the 22 February event, the beams in the East-West perimeter frames experienced hinging. However, as the North-South perimeter frames were previously hinged and softened, the torsional resistance expected from the overall system would have decreased. Consequently, the building might have a moderate level of torsional response (twisting clockwise on plan shown in Figure 29), which amplified the demand on the Northern East-West perimeter frames.

Due to the high beam depth-to-span ratio (850/2900), the beam elongation effects (geometrical elongation and plastic cycles cracking) were significantly more pronounced in the East-West perimeter frames. As expected, the elongation of beams created tension in the connection between the precast floors and supporting perimeter beams. The largest horizontal crack parallel to the double-tee flange support was approximately 20mm to 40mm wide. Slab mesh fracture was observed in floor topping close to the beam plastic hinges. In several locations at the Northern bays, the precast floors have dropped vertically about 10 to 20 mm, indicative of loss of precast floor seating support.

Beam-elongation effects on the integrity of the diaphragm action of precast flooring units with brittle wire mesh as topping reinforcing have been identified as a critical structural weakness well before the 22 February earthquake [20, 39].

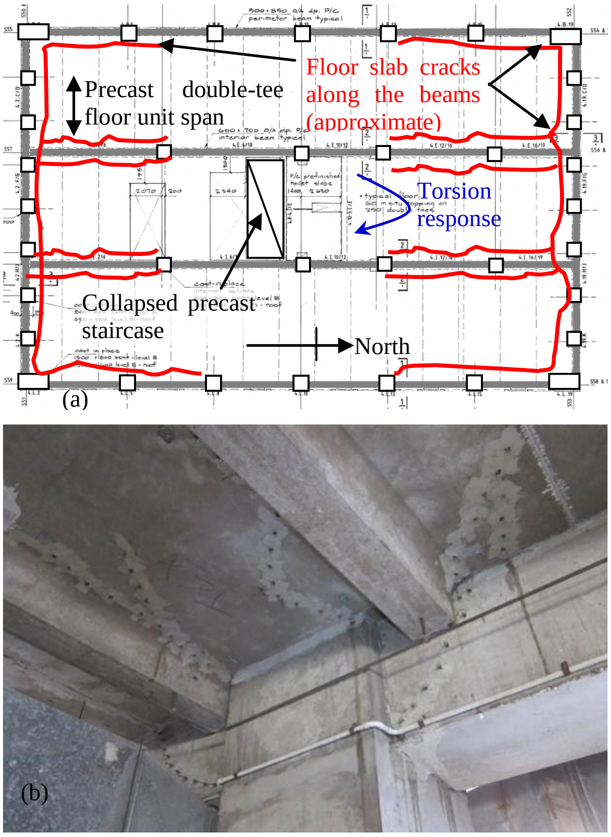

Figure 29: Ductile RC perimeter frames building with beams hinging, floor slab damage and collapsed precast staircase at upper levels (8th to 11th floor). Photo (b) is showing the crack repair done after the 4 September earthquake.

Figure 30: Extensive damage of floor diaphragm and loss of floor support for building in Figure 29 due to the beam-elongation effects of concrete frame inelastic response.

Plan and Vertical irregularity: There are a number of examples of modern RC frame-wall buildings where plan and vertical irregularity resulted in unexpected concentration of seismic demands on beams, walls and columns.

The Grand Chancellor Hotel (to be discussed in Section 6.3) is an example of the effects of plan and vertical irregularity on the overall lateral stability of the building.

Figure 31 and Figure 32 show an example of an 11-storey RC frame and wall building. The lateral resisting systems (frames and walls) are terminated at the ground floor level with the ground floor slab acting as a transfer diaphragm to the basement perimeter walls. 175 mm thick ground floor slab was reinforced with high-strength 12 mm diameter bars at 300 mm to 350 mm centres are provided (see Figure 32c).

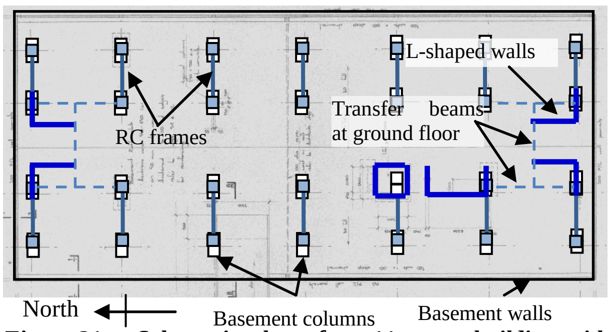

Figure 31: Schematic plan of an 11-storey building with plan and vertical irregularity resulting in severe basement columns shear-axial failure and transfer slab failure.

***Figure 32:** Vertical irregularity resulting in (a) severe basement columns shear-axial failure; (b) transfer beam repair and damage; c) Ground floor transfer slab failure.*

Four L-shaped 200 mm thick RC walls terminated at ground floor level and relied on a set of transfer beams (dashed lines in Figure 31) and slabs for inertia force transfer to the 150 mm-200 m thick perimeter basement walls. The 300x500 mm deep transfer beams were initially damaged in the 4 September 2010 earthquake and were repaired (see Figure 32b).

The basement columns (beneath upper columns and walls) were designed to be ductile gravity-dominated columns with well-confined but flexible section. Two separate columns were provided to reduce the flexural stiffness of the basement columns (Figure 32a).

There is also a plan stiffness irregularity, with the additional two sets of core walls on the Southern side. The plan irregularity resulted in torsional amplification and higher demand in the basement columns on the Northern side. Nearly all of the basement columns on the Northern side (first three gridlines) had suffered shear-axial failure (see Figure 32a).

The basement columns under the L-shaped walls were severely damaged. The transfer slabs between the L-shaped walls and the basement perimeter walls were also heavily damaged (see Figure 32c). The 11-storey building was at a 200 mm to 400 mm lateral lean (at the roof level) after the 22 February 2011 earthquake.

5.2 Modern (Post-1970s) RC Walls buildings

RC structural walls, or shear wall buildings were a relatively popular structural system for medium to high-rise buildings since the 1970s.

Perhaps due to the apparent increase in sophistication in design and structural analysis in recent years, a large percentage of the recently constructed RC walls was considerably thinner and more slender walls and with a minimum level of reinforcing and higher levels of axial load ratio. These walls, while detailed for flexural action, failed in brittle shear-compression or premature reinforcing tensile/compressive fracture, leading to an irreparable state of the buildings.

The high number of severely damaged modern RC wall buildings has indicated that the current design for slender RC walls with inadequate confinement steel outside the confined boundary zone, irregular shapes, or with inter-panel grouted (poorly confined) lap-splice is inadequate.

Wall web buckling - Figure 33 shows the overall buckling of one outstanding leg of a V-shaped (or L-shaped) shear wall in a 7-storeys building. The width of the buckled web was 300 mm, with an unsupported wall height of 2.66 m, resulting in a height-to-thickness (slenderness) ratio of 8.9. The boundary zone extended approximately 1.2 m into the 4 m long web. The boundary steel at the damaged end of the wall consisted of 16-24 mm deformed bars confined by 10 mm plain round bars at 120 mm centres, with a 180 degree hook on every other longitudinal bar.

The wall buckled over a height of approximately 1 m and crushing extended over 3 metres into the web. Horizontal cracks (approximately 1-1.5 mm width) were visible at the buckled end of the web, while inclined cracks in both directions at approximately 45 degrees were apparent in the middle of the web over the first storey height.

***Figure 33:** Seven-storey 1980s office block with significant compression failure of the V-shaped RC shear wall.*

The damage pattern described above and shown in Figure 33 suggests that the web may have initially experienced flexural tension yielding of the boundary steel, followed by buckling of the unsupported web over the relatively short plastic hinge length. The L-shaped cross-section would have resulted in a

deep compression zone with high compression strains at the damaged end of the web wall. Stability of the compression zone may have been compromised by a reduction in the web out-of-plane bending stiffness due to open flexural tension cracks from previous cycles.

Boundary zone bar fracture – Fracture of very light longitudinal reinforcement was also noted in modern high-rise buildings. In some cases (e.g. Figure 34), wide spacing of transverse reinforcement may have led to bar buckling prior to bar fracture. Bar buckling results in high localized strains at the location of bar bending and can decrease the tensile strain capacity at fracture. The architectural design of this building included numerous walls, making it possible to achieve the higher base shear required for a low ductility (nominal or limited ductile) structural system and thus avoiding the need for full ductile detailing.

Figure 34:* *Bar buckled and fractured in lightly reinforced slender RC shear wall: a) North-South Wall; b) East-West Wall.

Fracture of boundary reinforcement was also observed in the 200 mm thick wall shown in Figure 35. This 7-metre long wall (coupled with a 2-metre wall) was the primary E-W lateral force resisting system for an 8-storeys plus basement condominium. For the bottom four stories the wall was reinforced with 12 mm deformed bars at 100 mm centres in both directions, each face. The boundaries, extending 980 mm from each end, were confined with 6mm bar hoops at 60 mm centres, supporting at least every other longitudinal bar.

As shown in Photo 1 of Figure 35, fracture of at least four of the 12mm end bars occurred at the top of the ground floor. Core concrete generally remained intact in the confined boundary zone (except where fracture of bars occurred); however, crushing of the core extended into the unconfined web for approximately 3 m from the end of the confined region. The crushing in the web exposed spliced transverse bars, which could not contain the core concrete once the cover had spalled (Figure 35-photo 4). The damage in the web extended diagonally downward from the fractured boundary, suggesting that high shear stresses may have also contributed to the observed damage.

The building in Figure 35 also illustrates the risk of limited redundancy in the lateral-load resisting system. The secondary gravity structure, consists of light steel posts and beams, is unable to provide a lateral load redundant system.

Buckling failure of ducted splice – Figure 36 shows the buckling failure of grouted ducted splices for precast concrete wall at the Ground Floor level (above a multi-level basement). The ducts were meant to be grouted for anti-buckling confinement but in some cases, inadequate grouting was reported. The lack of cross-ties results in limited anti-buckling confinement after the spalling of the concrete.

Figure 36:* *Failure of unconfined grouted duct splice for longitudinal bars of precast concrete walls.

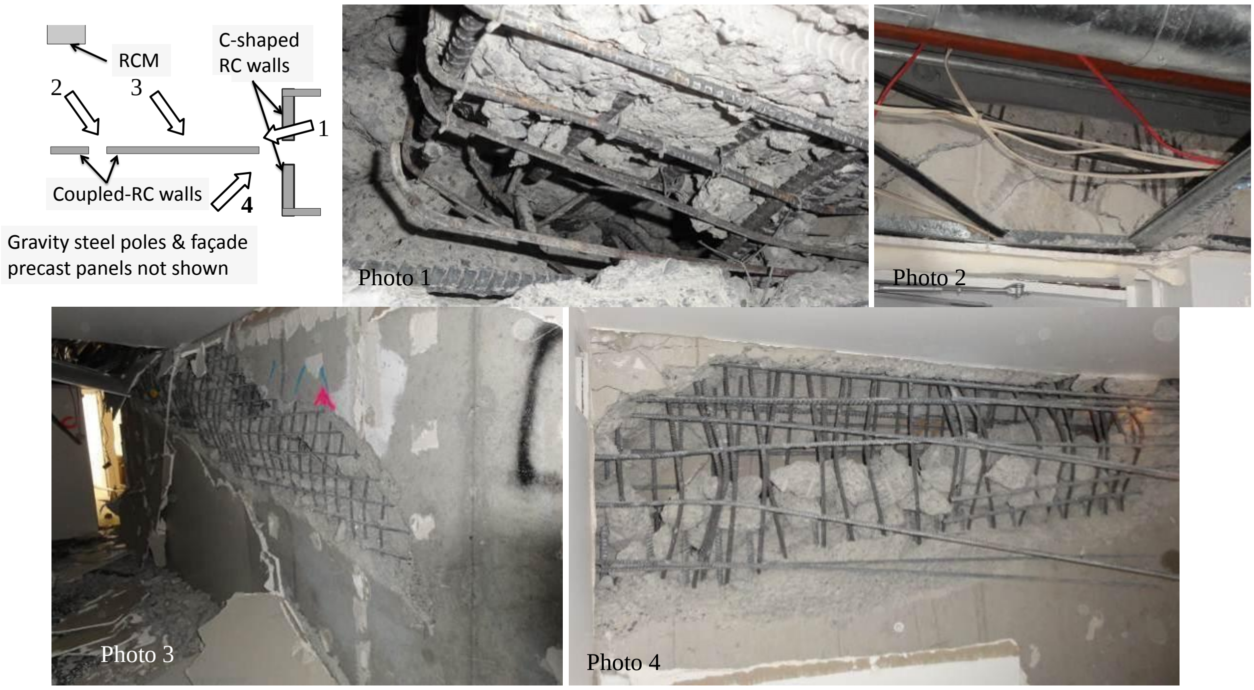

Figure 35:* *Boundary bar fracture and slender wall shear-axial failure in the Ground Floor of an 8-storey plus basement residential apartment building built in the 2000-2010.

5.3 Precast concrete connections and systems

Localised Corbel and Support Failure: Figure 37 shows one example where localised bearing failure resulted in a collapse of one-half of a car park floor of a 5-storey precast concrete building. The beam supporting the precast double-tee floor units fell from the supporting fin-shaped column and corbels, resulting in an approximately 800 mm drop of the supported floor. The corbel detailing may have resulted in the shear failure of the corbel support.

Figure 37: Localised collapse and loss of gravity support at the 1st floor at the 5-storey car park due to corbel failure.

Punching shear failure of post-tensioned slab: Post-tensioned concrete suspended slab are not widely used in Christchurch, possibly due to the negativity surrounding the post-tensioned slab system from the 1964 Anchorage Alaska earthquake. In the 22 February 2011 earthquake, a post-tensioned flat-slab on RC columns car park building, shown in Figure 38, pan-cake collapsed due to punching-shear failure of the post-tensioned slab.

Punching shear failure of the 220 mm thick flat-slab on wide columns (approximately 1200x450 mm) can be observed at the South section of the collapsed building. A section of the building over Dundas Street, consisting of in-situ prestressed RC beams had also collapsed, possibly due to progressive collapse initiated by the punching shear failure.

The post-tensioning in the slab did not pass through the columns. Forensic inspection of the collapsed columns suggests failure of limited continuity bars that were anchored into the beam-column joint.

No other post-tensioning anchorage or post-tensioned suspended slab damage failure was reported or known to the authors.

Punching shear failure of reinforced concrete flat-slab system was observed in one 10-storeys building designed and constructed in the 1970s.

Figure 38: Punching shear failure of a 5-storey post-tensioned flat-slab and columns building. (Photograph (c) is courtesy of David Swanson).

Lack of displacement-allowance for simply-supported elements: One consistent observation in the 22 February 2011 earthquake is the high displacement demands on structural elements. This applies also for "non-seismic – gravity-only elements such as simply-supported ramps, beams and staircases. Section 7 will expand further on the displacement incompatibility and demand on precast concrete staircases.

Single-storey car park ramps are typically constructed with simply-supported flooring units (e.g. precast concrete prestressed hollow core units or Hi-Bond steel-concrete composite deck). However, it was observed that often the seating and gap provided for the simply-supported ramp unit was insufficient to prevent unseating and/or pounding onto each other or into the abutments.

Figure 39 shows a column shear failure, possibly induced by the movement of the simply-supported ramp and trimmer beams. As the three parts (labelled A, B and C in Figure 39) all have different displacement responses (rigid to flexible in the order A to C), it is not surprising to see the damage in Figure 39.

***Figure 39:** Lack of displacement allowance for 'simply-supported' elements such as car park ramp leading to a column shear failure.*

Figure 40 illustrates the two observed failure modes (within the same car park complex as with Figure 39) of such simply-supported elements due to the lack of displacement allowance. Figure 40a shows an unseating of a long-span prestressed hollow core ramp/deck unit. This is possibly due to the failure of the supporting wall and the insufficient seating provided.

***Figure 40:** Lack of displacement allowance for 'simply-supported' elements leading to failure and collapse of car-park ramp: a) Unseating of hollow core unit at one-simply supported end; b) Collapse of one bay of ramp, possibly due to compressive buckling and pounding with the abutment. Photographs are courtesy of John Marshall [38].*

Figure 40b shows the collapse of one-bay of a ramp, possibly due to the compressive-buckling induced failure of the hollow core units as the deck/ramp pounded against the abutment.

The seismic gap and sliding joint in between the ramp units, and at the sliding support at the abutment should be increased as per the recommendation for simply-supported precast concrete staircases [11]. Furthermore, continuity reinforcement should be provided between the topping concrete and the prestressed hollow core ramp in order to limit delamination of the topping concrete [38].

5.4 Precast panels connection/anchorage failure

Failure and collapse of heavy precast concrete façade panels can be very hazardous to life-safety of the passer-byes. Further description of the performance of precast concrete façades can be found in a companion paper [4] in this special issue.

Figure 41 shows an example of a collapsed precast concrete panel due to the failure of the rigid connections at the two ends. One of the two collapsed panels (Panel B as indicated in Figure 41a-b), was rigidly connected to two separate buildings (which naturally have different displacement response). It is likely that Panel B was displaced due to the relative displacement of the two buildings, and hit the end of Panel A. It may explain why Panel A dropped one to two metres away. Figure 41c and d show the different 'rigid' anchorage connections used on the panels.

***Figure 41:** Failure of heavy precast concrete panel connections: a) Panels prior to the earthquake; b) collapse of the panel at the entrance; c) Close-up view of the two anchorage types; d) Pull-out concrete cone on the panel.*

Some precast concrete façade panel connection failures, as shown in Figure 42, occurred due to construction error. A close-up inspection (Figure 42b) of the connection angles attaching the concrete panels to the RC frame superstructure showed that the welding of the slotted bolt connection was welded to the washer plate.

This construction error would thus have restricted in-plane deformation of the concrete panels relative to the RC frame inter-storey drift. Consequently, the rigid "welded-slotted bolt" connection failed and the panels collapsed out-of-plane.

Figure 42: Failed precast concrete façade panels "welded and slotted-bolt" connection (construction error).

5.5 RC Tilt-up industrial/commercial buildings

Tilt-up precast concrete panels are a popular construction form for low-rise industrial/commercial buildings. The precast concrete panels are generally cantilevered at the base and joined together by steelwork or a concrete floor (for multi-floors) at the top. Shear connection between the panels is also typically provided.

Typical damage included fracture/failure of steel connectors and diagonal bracing, cracking of inter-panel connections and several complete collapses of the wall panels. Figure 43 shows a couple of examples of collapse/failure of precast concrete tilt-up structural walls.

Figure 43a shows tilt-up walls as a part of the lateral-load resisting system of a two-storey car park building in a suburb of Christchurch. The wall failed in-plane along the base, followed by a loss of anchorage to the 1st floor diaphragm, resulting in out-of-plane collapse.

Figure 43b shows an example of destabilisation and collapse of precast concrete tilt-up walls which were under construction at the time of the earthquake. It appears the connections between the orthogonal panels had failed, leading to the out-of-plane collapse of one panel and destabilisation of the other.

A more detailed report on the seismic performance of low-rise precast-concrete tilt-up structures is given in reference [32].

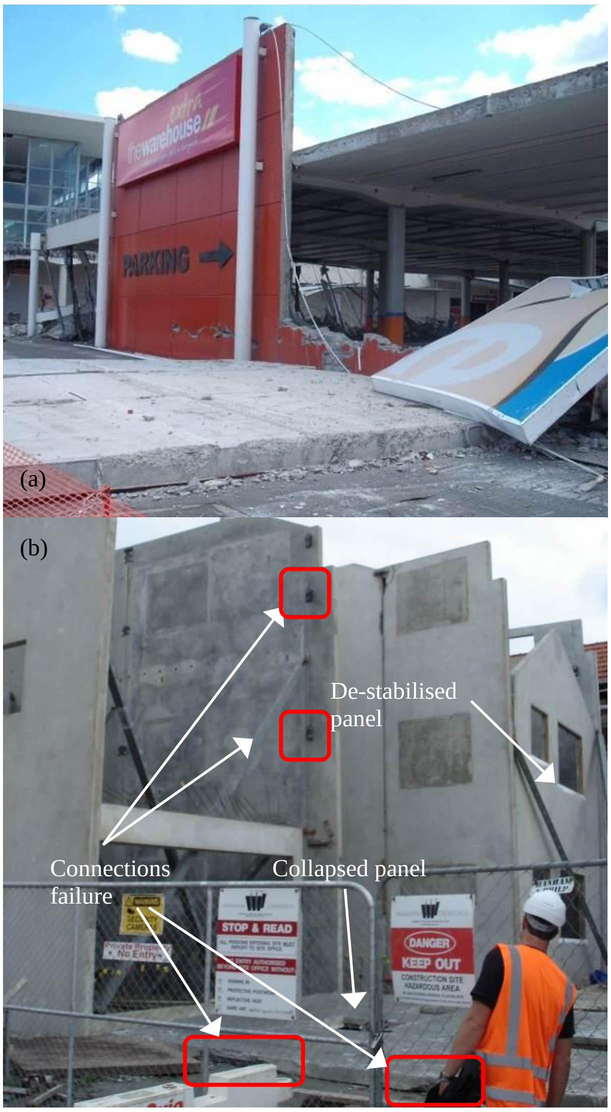

Figure 43: Collapse of precast concrete tilt-up structural walls: a) Localised flexural failure along the base of the wall panel; b) Destabilisation of tilt-up concrete wall under construction.

5.6 Advanced seismic resisting RC systems (post-tensioned PRESSS, supplementary damping and base-isolation)

The 22 February 2011 Christchurch earthquake has also tested a few innovative advanced seismic resisting RC systems such as the base-isolated moment-frame Christchurch Women's Hospital and the post-tensioned jointed-ductile precast concrete (PRESSS-technology) Southern Cross Hospital's Endoscopy Consultant Building.

The Christchurch's Women Hospital is an 8-storey RC frame and steel braced building, supported on 41 Lead-Rubber Bearing isolation devices (Figure 44a). The building came through all the Canterbury earthquakes without significant structural damage in spite of some clear evidence of lateral deformation demand at the base relative to the surrounding ground [24]. The observed deformation at the building boundary (e.g. Figure 44b) suggests the lateral deformation to be at least 100 mm in the 22 February event.

While after the 4 September earthquake the isolators have a residual displacement of 25 mm, the isolators had, incidentally returned to its original position after the 22 February aftershock (note the near zero residual displacement shown in Figure 44a).

The current increased requirement for the design level of seismicity for Christchurch [16] (i.e. a Hazard Factor, Z of 0.3 instead of 0.22) is only valid for building with fundamental period up to 1.5 s. For building period above 1.5 s, special study of the seismic demand is required. The limitation was in response to the high spectral acceleration amplification in the long period range (2.0 s to 3.0 s) as discussed in Section 2.1. Such long period amplification might result in large boundary displacement gap requirements and stronger isolated superstructure with less reduction in the superstructure design base-shear.

Figure 44:** **a) A Lead-Rubber-Bearing isolation device with near zero residual deformation (compared to 50mm after the 4 September earthquake); b) The seismic moat cover on the ground level indicates significant lateral movement during the earthquake.

The four storeys Southern Cross Hospital's Endoscopy (SCHE) Building is the first South Island PRESSS-technology building with precast concrete un-bonded post-tensioned frames (North-South) and coupled-walls (East-West) [58].

The beam-elongation effect on the floor diaphragm from the post-tensioned frames was mitigated by placing the precast floor units orthogonal to the rocking moment-resisting frames and by using cast-in-situ band beam-slab at the top-hinging-only beam-to-column rocking interface (Figure 45c). This is achieved by having only beam top longitudinal reinforcement connected into the column, in addition to the post-tensioned tendons.

No observable structural damage was detected in the building after the 4th Sept 2010 7.1 M_w Darfield earthquake. SCHE building was almost immediately re-occupiable (after a prompt structural assessment).

In the 22nd Feb 2011 6.2 M_w Christchurch earthquake, the structure had signs of significant transient movements, especially in the East-West longitudinal direction (consistent with the polarity of the Feb earthquake). On the top of the south walls, very minor crushing of the cover concrete was observed at the interface between the coupled walls. Most of the U-shaped Flat Plates (UFPs) had Lüders yield lines (Figure 45b), indicating the building's inter-storey drift exceeded 0.5%-0.75% (corresponding to the yield drift of the UFPs).

Preliminary non-linear time-history analyses of the Endoscopy Consultant Building seismic response under the 22 February earthquake [58] suggests the building has experienced at least 2.5% inter-storey drift demand. Minor cracking of the internal Gib-lined partitions also indicates significant level of transient lateral deformations of the building.

As a reaction to the costly repair and demolition of many conventional RC buildings, the concept of designing for damage-avoidance systems using seismic-isolation, supplementary damping, or the re-centering rocking PRESSS system is emerging [10, 69].

Given the suddenly appreciated importance of damage-control design and also the cost-efficiency of such systems, the post-earthquake reconstruction of Christchurch may see more implementation of such advanced seismic resisting systems.

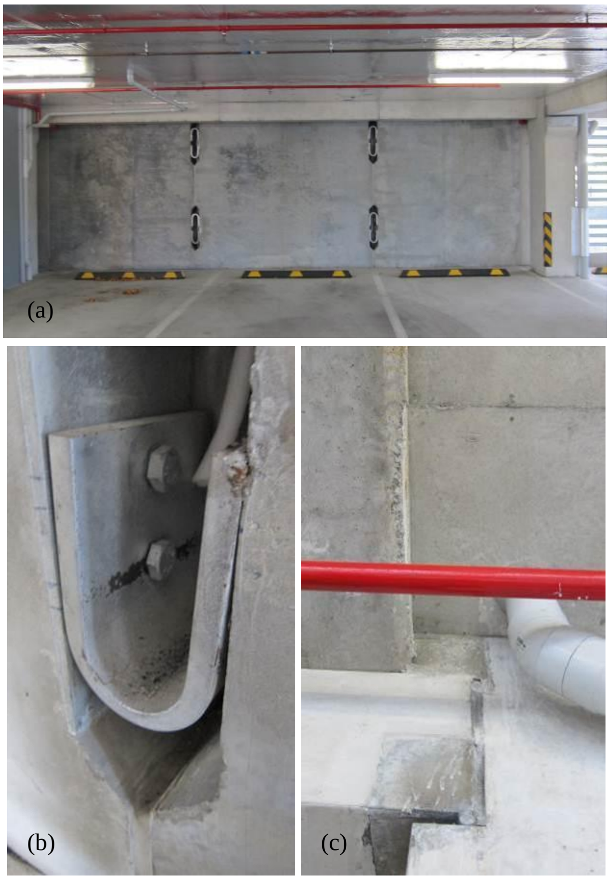

Figure 45:** **Self-centring precast concrete system implemented for a newly constructed private hospital facility: a) Coupled post-tensioned rocking walls; b) Yield lines observed in the U-shaped flexural plates coupling the post-tensioned rocking walls; c) No residual crack along the rocking interface at the beam-column connection.

6 CRITICALLY DAMAGED OR COLLAPSED RC BUILDINGS

In response to the public concern about the damage to and collapse of major buildings resulting in significant fatalities, the New Zealand Government, through its Department of Building and Housing (DBH) initiated a technical investigation on the structural performance of the four large multi-storey buildings in the Christchurch CBD which failed during the 22 February 2011 Mw 6.2 earthquake. The buildings included in the investigation are the Canterbury Television Building (CTV), Pyne Gould Corporation Building (PGC), Hotel Grand Chancellor (HGC) and Forsyth Barr Building.

The Part 1 Expert Panel Report [18] along with technical investigation reports on three of the buildings (PGC, HGC and Forsyth Barr Building) have been submitted to the *Royal Commission of Inquiry into Building Failure caused by Canterbury Earthquakes* [70]. The following sub-sections describe some of the structural characteristics and the observed damage/response of these buildings. Interested readers should refer to the Royal Commission of Inquiry website [70] for the more definitive and extensive reports on these buildings.

6.1 Pyne Gould Corp (PGC) Building

A summary of the building structural characteristics and observed damaged is discussed below; the technical

investigation report conducted for the DBH should be consulted for further details [5].

The Pyne Gould Corp (PGC) building was designed and built in 1963-64, near the time of seismic code revision in 1964-65 (NZS1900:1965 [44]). It is a six-storeys five-by-five bays RC frames building (Figure 46) with an internal core wall. Figure 47 shows the typical upper floor's structural plan view.

Figure 46: ***Pyne Gould Corp (PGC) Building photographed from the South-East elevation after the 4 September 2011 earthquake.***

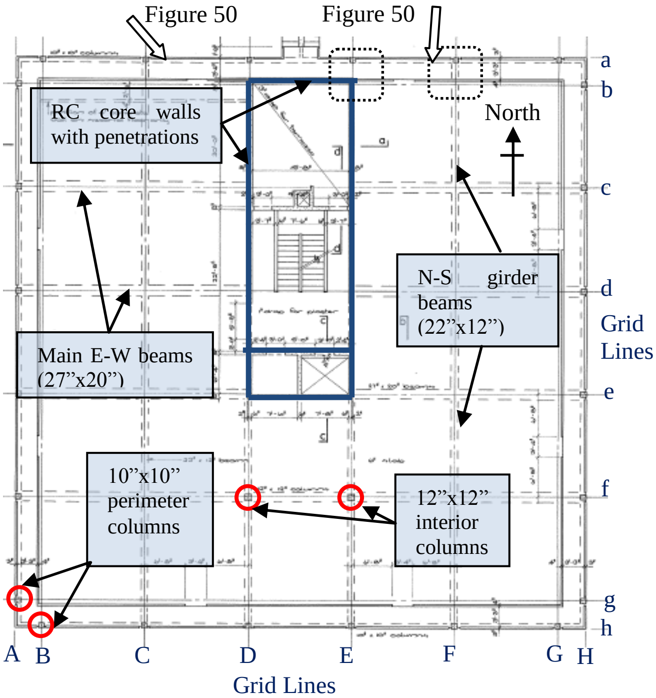

Figure 47: ***Plan view of the typical upper floors (2nd to 4th Floors).***

Structural systems: The lateral load resisting system consists of 8" (200 mm) thick RC core walls with two 15 m long RC walls along Grid Line D and E (acting in the North-South direction), and three shorter (two 5 m and one 2.6 m long) RC walls along Grid b and e (acting in the East-West direction). Figure 48 shows the East-West cross elevation view, which indicates some of the openings in the North-South 15 m long walls. The shorter RC walls have significant openings (two door openings of approximately 850 mm x 2,200 mm dimensions).

In general, the 200 mm thick RC walls are very lightly reinforced with a single layer of 5/8" (16 mm) diameter deformed reinforcement spaced vertically and horizontally at 15" (380 mm) centres. Longitudinal bars are lapped at above the floor level, with a lap length of 20" (508 mm). No wall cross-ties or boundary confinement ties are observed on the drawings, which was typical for RC walls of this vintage.

In the East-West direction, six RC three-bays (10m-5m-10m) frames (most likely designed for gravity-load only) would contribute a minor level of lateral strength and stiffness. The E-W direction main beams are 33"x24" (840x 610 mm) at the 1st floor and 27"x20" (685 x 510 mm) at 2nd floor to roof. At the Northern side of the building, four of the RC frames are framing into the core walls. In the second Southern frame line, there are two interior columns, measuring 16"x16" at the ground floor, and 12"x12" at the upper levels (see Figure 47).

In the transverse North-South direction, there are two perimeter RC five-bay (5 m bay length) frames with no interior columns/framing. The transverse girder beams are 33"x24" (840 mm x 610 mm) at the 1st floor and 22"x12" (560 mm x 305 mm) at the upper levels. The transverse beams span a regular length of 5.08 m. At the perimeter of the building, there are 38"x6" (965 mm x 150 mm) edge beams.

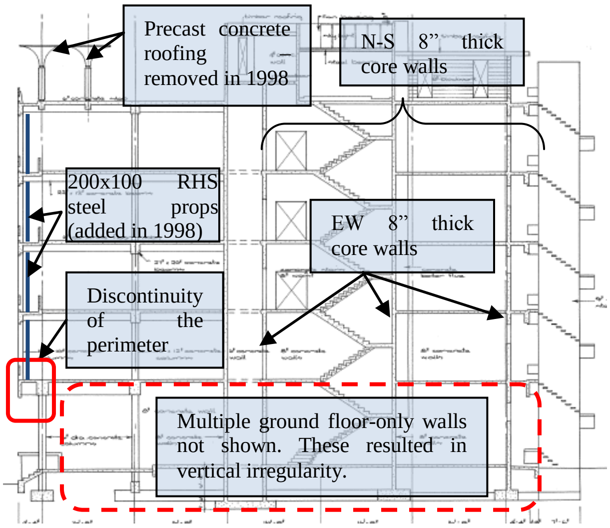

Figure 48: ***North-South elevation on Grid Line D.***

The ground floor perimeter columns are 16" (400 mm) diameter circular with 1/8" (3.2 mm) thick steel encasing while at upper levels, the perimeter columns are 10"x10" (254 mm x 254 mm). The perimeter columns have a distinct discontinuity at the 1st floor (between Level 1 and Level 2 columns). The upper level perimeter columns are offset 52" (1.321 m) from the Level 1 (Ground floor) columns. Steel beams were used at the connections between the upper floor perimeter columns to the ground floor columns.

The columns are generally lightly confined and poorly detailed for deformation, when compared with what the current code [48] would require. Above the ground floor columns, ¼" (6.5 mm) diameter stirrups at 9" (230 mm) centres are typically provided uniformly along the whole column height. ¼" (6.5 mm) diameter spiral ties at 9" (230 mm) pitch are used for the ground floor columns. No joint transverse reinforcement was provided.

After a seismic structural review in 1997, 18 200x100RHS steel props were installed behind each perimeter column (see Figure 48). Several precast concrete roof canopies were removed to reduce the falling hazard.

As this is a building built prior to the introduction of modern seismic codes in the mid-1970s, the building had several critical detailing and reinforcing deficiencies typical of that vintage (lightly reinforced walls, no boundary or confinement reinforcing for walls, lack of beam-column joint reinforcement, limited number of walls, inadequate column's and beam's lap-splice length and inadequate floor/beam to column/wall anchorage) that could contribute to the collapse.

There is also a vertical stiffness and strength irregularity in between the Ground Floor and the upper floors, as there are several ground floor RC walls that discontinued at upper levels.

According to the DBH report on PGC [5], the building had suffered minor damage after the 4 September 2010 and 26 December 2010 earthquakes. Minor diagonal cracking of the RC core walls was observed and the occupants noted "the building became more responsive" in the subsequent aftershocks prior to the 22 February 2011 earthquake.

Damage observed in the 22 February 2011 earthquake: The upper five storeys suffered a soft-storey pancake collapse, with collapsed floors slanting towards to the East side, indicating of soft-storey failure along the East-West direction (Figure 49). No evidence of torsional twist was observed from the collapsed building. The ground floor structure appears mostly intact.

***Figure 49:** a) Southern elevation of the collapsed PGC building; b) South-Eastern elevation of the collapsed PGC building.*

According to the DBH report [5] and observed damage, the collapse appears to have been initiated at the 1st and 2nd Floors as shown in Figure 49 and Figure 50. The RC core walls in between the 1st and 2nd Floors had collapsed (Figure 50-Zoom A). The core walls at the upper floors were generally intact. Some diagonal cracks can be observed at the 2nd floor section of the RC walls.

Considering the limited shear capacity of the 200 mm thick short walls in the East-West direction, the long RC walls are relied upon to provide the majority of the overturning moment. The 200 mm wall with single layer of vertical reinforcement has only limited ductility capacity and likely failed in flexural compressive buckling. The wall section is not confined for significant ductility demand.

The RC frames were generally unable to develop ductile beam-hinging due to the evident lack of capacity design principles and poor connection detailing. The beam-column joints and columns failed prematurely (Figure 50-Zoom B and C). The frames were unable to sustain the significant displacement demand (after the failure of the RC core walls).

***Figure 50:** Various failure mechanisms observed on the Northern elevation of the collapsed PGC building.*

The beam-column joints were not reinforced with transverse ties and appeared to fail in shear. Column longitudinal bars were buckled at the damaged beam-column joints, losing theirs gravity-load carrying capacity. As shown in Figure 50-

Zoom A, 1st and 2nd Floor RC columns were detached from the beam-column joints and lost theirs vertical-load capacity. Pull-out and anchorage failures of beams were also observed (Figure 50-Zoom A and B).

The pull-out anchorage failure of the connection between the core walls and the framing beams and slabs is observed at least in the upper 2-storeys (Figure 49b and Figure 50-Zoom B). This is likely to occur with significant rotational demand at these connections due to the failure of the frames and walls.

6.2 Canterbury Television (CTV) Building

The actual cause of failure that led to the brittle and catastrophic collapse of the CTV building is currently under investigation by the DBH-commissioned technical study and the Royal Commission of Inquiry [70]. The following paragraphs are our general observation based on the available information and forensic inspection. Interested readers should follow the outcomes of the DBH and Royal Commission inquiries [70] to gain further understanding of the critical structural weaknesses that lead to the unexpected collapse of this mid-1980s-designed building.

Figure 51 shows the CTV building from the south-east elevations. The typical floor plan of the CTV building is presented in Figure 52.

Figure 51: ***The CTV Building from the South-East corner. Photograph is courtesy of Dr Yuji Ishikawa.***

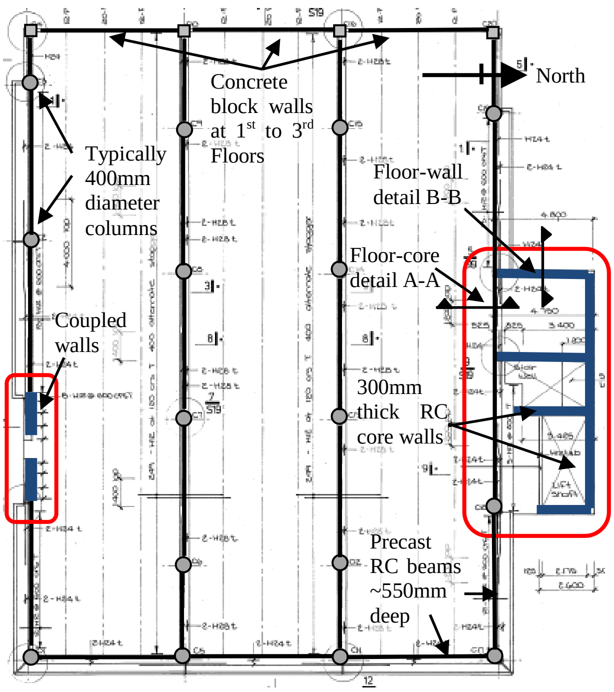

Figure 52: ***Typical floor plan of the CTV building.***

The 6-storey RC building comprises a coupled-shear wall on the Southern side and a core RC wall on the Northern side of the building. Four RC frame lines provided some lateral resistance in the East-West direction. The entire building, with the exception of the core wall collapsed during the 22 February 2011 aftershock. A major fire broke out almost immediately after the collapse of the building.

The 300mm thick RC core walls on the Northern side of the building, measuring 4.8 m x 11.5 m long, were generally well-reinforced with ductile detailing typical of 1980s construction. However, the RC core walls had limited connections to the floor diaphragm of the building, with approximately 11.5 m length of floor-slab (minus some void area due to lift penetration).

Figure 53 illustrates the typical slab-to-core walls (slab and wall) connection detail. A Hi-Bond steel deck with 200 mm thick concrete reinforced with one-layer of cold-drawn wire mesh and one layer of H12 bars at 200 mm centres was relied upon for transferring the seismic inertial load from the main structure to the RC core walls.

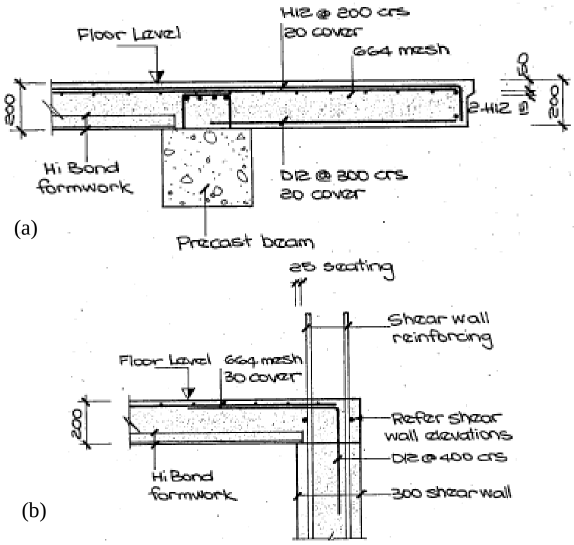

Figure 53: ***Structural detail of the diaphragm connection to the RC core walls (refer to Figure 52): a) Slab – core wall connection A; b) Slab walls connection B.***

Figure 54: ***Northern RC core walls of the CTV Building. Photograph (a) is taken on the 23rd February 2011 by Mark Mitchell published in New Zealand Herald. Photograph (c) illustrates the remains of the Hi-Bond floor slab deck at 3rd and 4th Floors.***

Figure 54a shows the collapsed CTV building with the Northern RC core walls predominantly intact. The RC walls did not exhibit any significant residual distress or cracking, as observed in the post-earthquake inspection. It appears the main framed-superstructure detached from the RC core walls under the severe earthquake shaking.

On the Southern elevation, there was a pair of 2.05 m long 300mm thick RC walls coupled with a 900mm long coupling beam that would provide significant lateral load resistance. These coupled-walls remained largely intact after the building collapse (see Figure 55), with only limited cracking observed in the ground floor coupled-wall. The 1st Floor walls was observed to sustain significant out-of-plane deformation demand, possibly arising from the collapse.

It appears only limited reinforcing was provided between the slab-to-coupled-wall connection (Figure 55c). Furthermore, the drawings indicate the H12 bars at 600 mm centres and the floor wire-mesh were not anchored using 90-degree bent hooks, typical of such connections (in modern RC design).

Figure 55: Coupled walls on the Southern side: a) the coupled walls remains intact on the Ground Floor with limited flexural or shear cracking; b) All six pairs of the coupled-walls were accounted for during a post-demolition inspection – limited damage were observed on these walls; c) the connection detail of slab-to-coupled walls.

The building comprises four RC frames in the East-West direction and two frames in the North-South direction. It appears these frames are predominantly gravity-load carrying frames.

The typical columns are 400 mm diameter RC columns with six distributed HD20 (20 mm diameter) longitudinal reinforcement. The columns had 6mm spiral reinforcing at 250mm pitch. The typical beams are 400x550 mm deep precast concrete beams with closer stirrup spacing near the supports than provided for the interior beams.

All of the RC frames collapsed during the 22 February 2011 6.2 M_w main shock. Many of the beam and column elements were found 'intact' in the preliminary post-demolition forensic inspection of the building site (Figure 56).

Figure 56: Post-demolition inspection of the RC frame elements: a) RC column with R6 spiral ties at 250 mm centre and six-HD20 longitudinal reinforcement.

6.3 Grand Chancellor Hotel (GCH)

The 22-storey Grand Chancellor Hotel (GCH) (1970s parking structure + 1986-1988 hotel tower construction) was severely damaged during the 22 February 2011 earthquake, leading to an approximately 1,300 mm horizontal lean of the top of the tower and restricted access to the potential fall zone around the building (Figure 57).

A summary of the building characteristics and response during the earthquake is provided below; an extensive study conducted for the DBH [19] and the Royal Commission of Inquiry [70] should be consulted for further details.

Significant structural irregularities influenced the behaviour of the GCH building in the 22 February earthquake. Most notably the east side of the building (bay D-E) was cantilevered over Tattersalls Lane (Figure 58), which was a subsequent redesign due to unexpected legal issues.

The building was constructed in two phases. The lower 7 (or 14 half-height car park) storey structure, which comprises RC shear walls and cast-in-place flat slabs and columns, was constructed first. The upper 15 full-height storey structure,

which comprises of perimeter moment frames with a precast floor system, was added subsequently.

As indicated in Figure 58, the Eastern bay of the lower 14 half-height floors was cantilevered using several very deep transfer girders between levels 12 and 14. The southernmost transfer girders were supported on a critical shear wall denoted as D5-6 in Figure 59. Above the 14th floor, bay D-E is cantilevered by beams at each level and grid line.

Figure 57: The Southern elevation of the Grand Chancellor Hotel, with a distinct 200 to 400mm lean towards the East (right) side immediately after the 22 February 2011 earthquake.

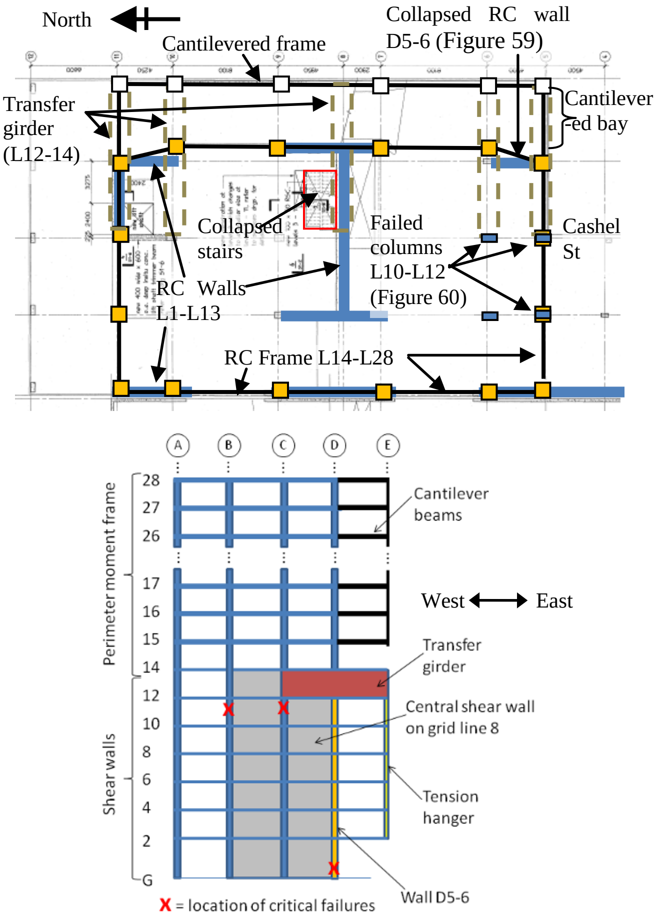

Figure 58: Schematic plan and elevation of the Grand Chancellor Hotel. The floor numbering is based on the original construction drawings – the lower 14 floors are half-height car park floors. The building comprises 22 suspended storeys which includes a plant room level.

The 5 m long 400 mm thick RC wall D5-6 on the Southern side, supports a disproportionately large tributary gravity load from all floors as a result of the cantilever system. Wall D5-6 was reinforced with two layers of 20 mm diameter vertical bars at 300 mm centres and two layers of 16 mm diameter horizontal bars at 200 mm centres. The wall boundary reinforcement consisted of 4-D24 supported by a single plain round 10 mm diameter hoop at 150 mm centres. The symmetrical wall on the Northern side was more heavily reinforced (one bar size up and more boundary reinforcing) as it has a lower "design" axial force level compared to wall D5-6.

As shown in Figure 59-left, during the 22 February earthquake wall D5-6 experienced a brittle shear-axial failure at its base and displaced downward approximately 800 mm along a diagonal failure plane through the thickness of the wall. The failure plane, extending the full length of the wall, appeared to initiate at the top of the lap splice in the web vertical reinforcement. The limited hoops in the boundary appeared to have opened allowing the boundary longitudinal bars to deform with the shortening of the wall. Crushing of concrete was also noted at the top of the lobby wall, likely to accommodate the out-of-plane movement of the wall as it slid down the diagonal failure plane.

Wall D5-6 was likely supporting very high axial loads from several sources. First, as noted previously the wall supported a disproportionately high tributary area due to the cantilever structure. Secondly, the corner column of the upper tower perimeter moment frame would have imparted high axial loads due to overturning moments, particularly with any bi-directional movement to the south-east. Thirdly, vertical excitation of the cantilever structures, both above and below level 14, could have exacerbated the axial load on wall D5-6. Finally, wall D5-6 would have also attracted in-plane loads due to N-S earthquake excitation, leading to flexural compression stresses on one end of the wall.

Considering the potential for simultaneous compression from all sources of axial loads described above, it is expected that the combined axial load and bending in the wall likely exceeded the concrete compression strain capacity given the limited tie reinforcement provided at the base of the wall. It is noted that wall D5-6 was relatively more slender for its double-height at Ground Floor. The double-height atrium may result in wall aspect ratio (height-to-thickness) that was not code-compliance [19].

Some out-of-plane drift of the wall during the earthquake excitation and the plane of weakness created at the end of the splice of the web vertical reinforcement, further contributed to the location of failure at the base of wall D5-6. It is likely that failure of wall D5-6 precipitated other significant damage observed in the building, including shear and axial failure of level 10 columns supporting the southern transfer girders (see Figure 58 and Figure 60), lap splice damage where the tension column connected to the transfer beam on grid line 8, and hinging of beams on east face of the building (Figure 59-centre).

With the failure of wall D5-6, the two columns at level 10 immediate below the Southern-end transfer girders (see Figure 58) are likely to have experienced significant and a sudden increase in force and deformation demands. Axial loads would have increased as gravity loads redistributed with the axial failure of wall D5-6. Shear demands would have increased as the columns provided a partial moment restraint for the transfer girders. Finally, progressive (albeit instantaneous) failure of the wall D5-6 and the columns under the transfer girds also resulted in shear-failure of the next line of columns on Grid B.

Similar to many buildings in Christchurch, the GCH building had two sets of precast staircases, back-to-back in scissor alignment, located at the centre of the building adjacent to the primary E-W shear wall.

The precast concrete scissor staircases were supported by cast-in-situ transverse RC beams, spanning in between two interior RC frames. The shear and bearing transfers were achieved by two 120-140 mm long protruded 76x76x6.3mm RHS. The available seating was approximately 70mm, considering construction tolerance and the available 30mm gap.

The significant lateral deformation of the building and the localised vertical collapse at the South-East corner of the building would have imposed substantial differential displacement between the supporting beams of the staircase. The excessive differential lateral deformation resulted in the pull-out failure of the RHS stubs and resulted in progressive collapse of the precast staircases. Whether this detailing is the critical weakness of the collapse of one of the two internal staircases in GCH (see Figure 65), whose lateral displacement demand were exacerbated by the failure and tilting of the base wall, will need to be further investigated.

7 STAIRCASES IN MULTI-STOREY BUILDING

Collapse and severe damage of staircases in multi-storey buildings have been observed in many instances in the 22 February 2011 earthquake.

The concern of 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 was also noted and reported after the 4 September earthquake [35].

In a number of medium to high-rise buildings, staircases exhibited significant damage in buildings where the inter-storey movements of the staircases have been restrained. Complete or partial collapses of internal precast concrete staircases have been reported for at least four multi-storey high-rise buildings (e.g. Figure 61 to Figure 65).

Minor to moderate levels of movement/damages of the staircases were observed in many other mid- to high-rise buildings (Figure 62).

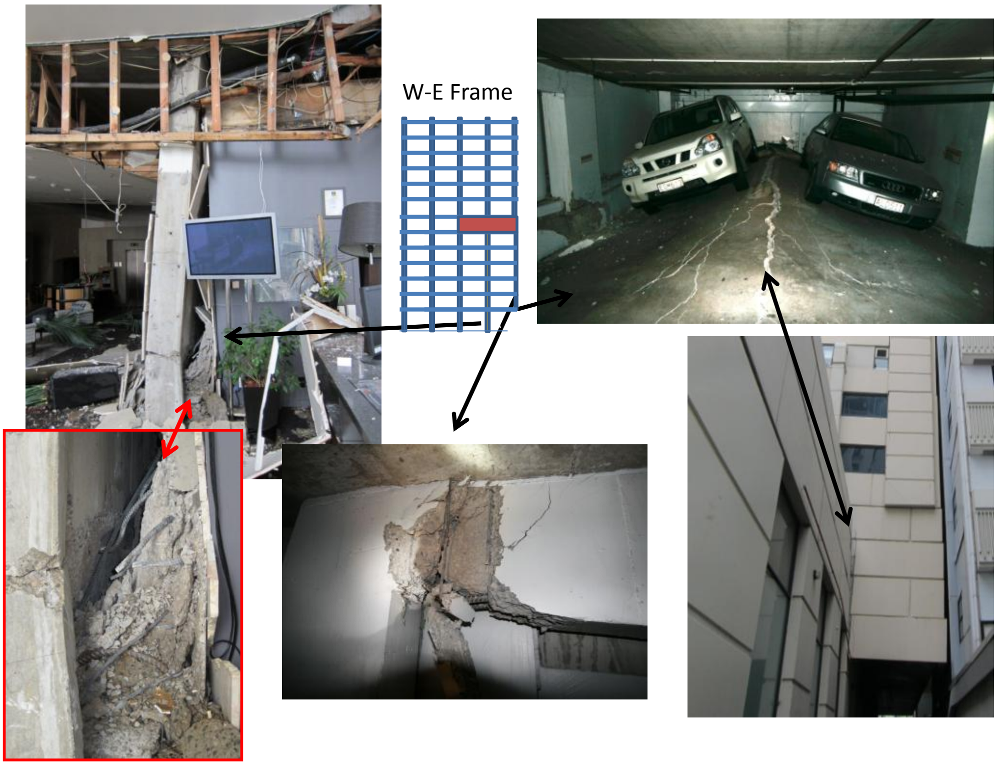

Figure 59: ***Grand Chancellor Hotel: The shear-axial failure of the RC wall D5-6 and the resulting damage pattern.***

Figure 60: ***Grand Chancellor Hotel: The shear-axial failure of the RC columns below the transfer girders at Level 10 and 11.***

Figure 61: Collapse of precast concrete staircase in multi-storey buildings.

As discussed in Section 2.2 and elsewhere in this report, one consistent observation in the 22 February 2011 earthquake is the very high displacement demands on structural and non-structural elements. The observed staircase damage in the multi-storey buildings indicated that the deformation allowance they had been designed for (even when compatible with the code-requirements at that time) was typically inadequate to sustain the very high seismic demand.

Considering that staircases are a critical safety egress in buildings, it is clear that a major re-consideration of the design philosophy of staircases in multi-storey buildings (RC or otherwise) will be needed. An interim approach to assess and retrofit existing stairs has been developed and issued as Practice Advisory by the Department of Building and Housing [11, 17]. Further description of New Zealand practice for staircase design is given in [11, 34].

Figure 62: Typical 'severe' top and bottom landing damage of precast concrete staircase in multi-storey buildings.

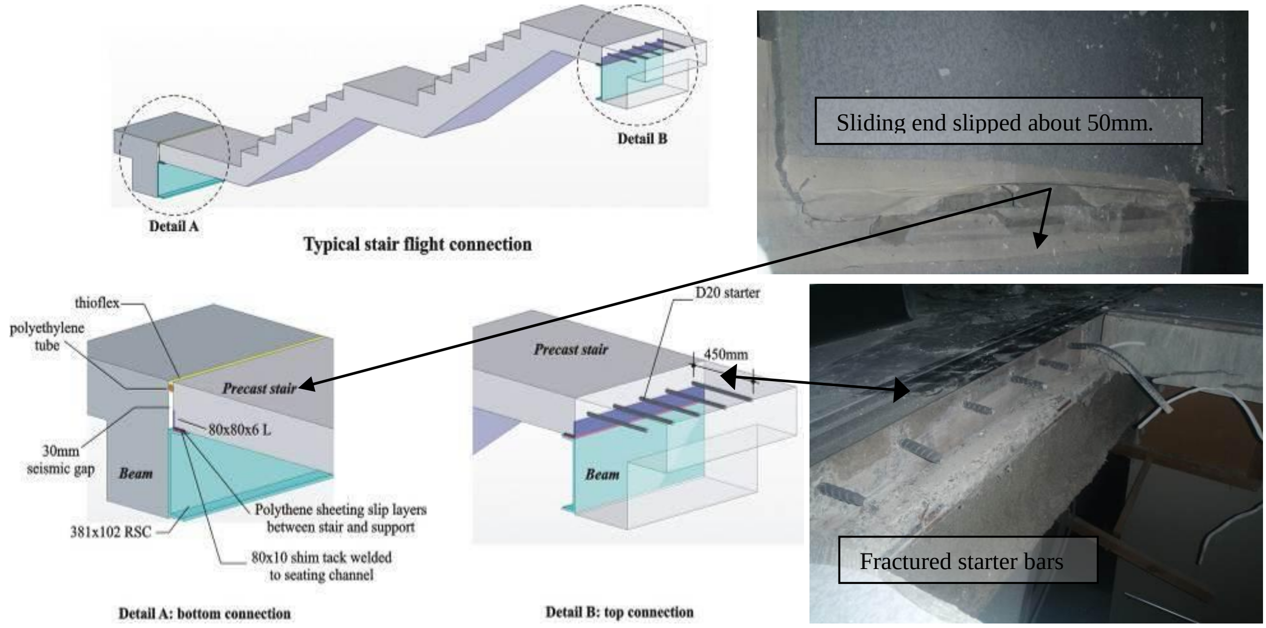

***Figure 63:** Typical detailing and damage of staircase with partially pinned-slide bottom connection; cast-in-situ connection at top with longitudinal starter bars lapped at landing. Image (left) is courtesy of Umut Akguzel and photographs (right) are from USAR engineers.*

***Figure 64:** Collapse of one out of two internal scissor staircases in a multi-storey RC frame building. The staircase was under repair work for the damage sustained in the 4 September 2010 earthquake.*

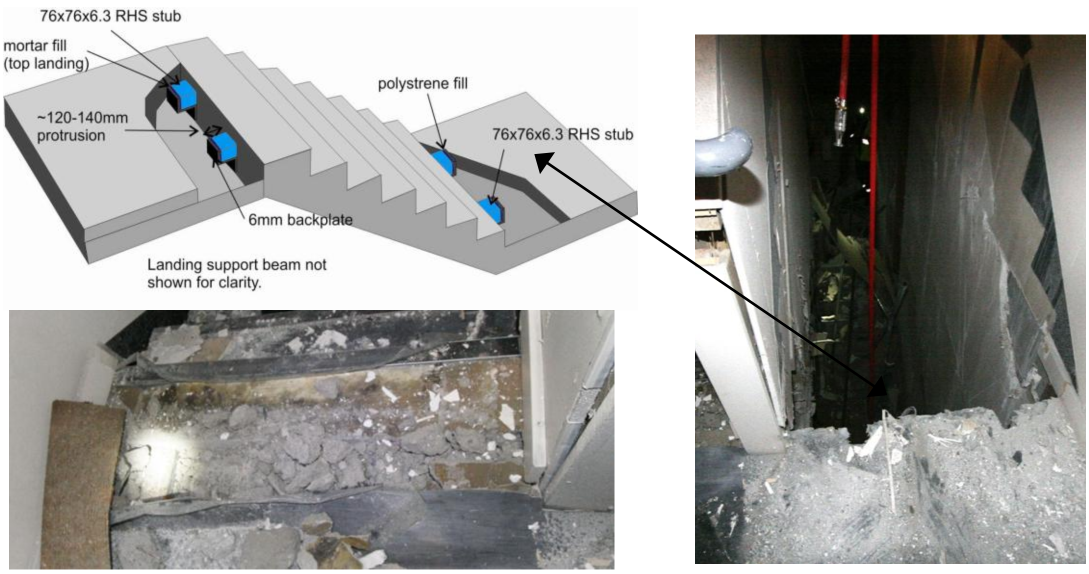

***Figure 65:** Alternative typical detailing of staircase- Type A - Pinned-slide connections with RHS shear keys on both ends and observed failures. (Damage photographs are courtesy of USAR engineers).*

8 EMERGENCY AND POST-EARTHQUAKE REPAIR

Immediately after the 22 February earthquake it was recognized that several RC buildings had suffered critical damage, bringing into question the stability of the buildings during on-going aftershocks. Rapid stabilization techniques were needed to ensure public safety and facilitate response and recovery efforts in the immediate vicinity of the buildings.

Before stabilization methods could be selected and designed, it was essential to determine the extent of damage to the structures and the probable cause. Critical to this process was the availability of structural drawings for the damaged buildings; facilitating rapid evaluation of the probable extent of damage prior to detailed inspection of the damaged buildings. Critically-damaged buildings were also monitored by surveyors to determine post-earthquake residual deformations and any further deformation with subsequent aftershocks.

The first priority of the emergency repair was to achieve sufficient stability such that emergency workers' access to surrounding streets and buildings was considered safe. Sufficient stabilization to enable escorted access to the interior of the damaged building for the recovery of important contents was a secondary objective. It was generally not anticipated that the emergency stabilization would lead to a condition where the building could be re-occupied. Many of the buildings which received emergency repairs are expected to be demolished in the coming months.

Figure 66 through to Figure 69 provide examples of emergency stabilization techniques used within the first weeks of the 22 February earthquake.

Figure 66 shows the stabilization of shear wall D5-6 from the Grand Chancellor Hotel discussed in Section 6.3 (see Figure 59-left for condition of wall prior to stabilization). A primary design consideration in selecting this stabilization technique was to limit the time the contractor would be in the building prior to completing the concrete pedestal in the first stage of construction.

***Figure 66:** Concrete encasement for temporary stabilisation of an axial-shear-damaged RC wall.*

The concrete pedestal was mass concrete with minimum reinforcement cast around the severely damaged wall base to ensure further crushing or movement of the wall could not occur. After the pedestal was completed, the stability of the building was considered to be dramatically improved and the contractor was allowed extended access to the building. Reinforcement was placed along the wall height and under the damaged slab prior to completing the stabilization by shotcreting both sides of the wall, with sufficient anchorage to the 1st floor slab soffit.

Figure 67 shows a typical stabilization technique used to restore the axial and shear integrity of several heavily damaged columns (see Figure 60 for condition of columns prior to stabilization). The steel encased reinforced concrete improves confinement and shear capacities of the damaged RC columns. The steel jacket was fabricated in several sections so it could be easily moved into the building, connected together around the columns, sealed at the joints between sections, and finally filled with concrete.

***Figure 67:** Emergency stabilisation repair of the columns with axial-shear failure. See Figure 60 for the pre-repair condition of the columns.*

Figure 68 shows an example where the steel-encasing reinforced concrete jacketing was used in conjunction with a concrete pedestal. The emergency stabilisation shown in Figure 68 corresponds to the building damage discussed in Figure 31 and Figure 32.

***Figure 68:** Emergency stabilisation repair of the basement columns with axial-shear failure.*

The steel encased RC jacketing provides improved confinement and shear capacities to the damaged columns. The concrete pedestal was expected to provide additional base-shear capacity (considering the ground floor diaphragm was no longer effective in transferring shear forces to the basement perimeter walls).

For cases where damage had not significantly impacted the stability of the building, but where extensive concrete crushing and/or bar fracture made standard repair techniques (e.g. epoxy injection) insufficient, encasement of the damaged region in reinforced concrete was typically adopted.

Figure 69 shows the repair to the damaged RC walls from Figure 34. In Figure 69a, bolted steel straps and a U-shaped confinement plate were provided in order to restore the confinement capacity of the wall with buckled longitudinal bars (and inadequate confinement ties).

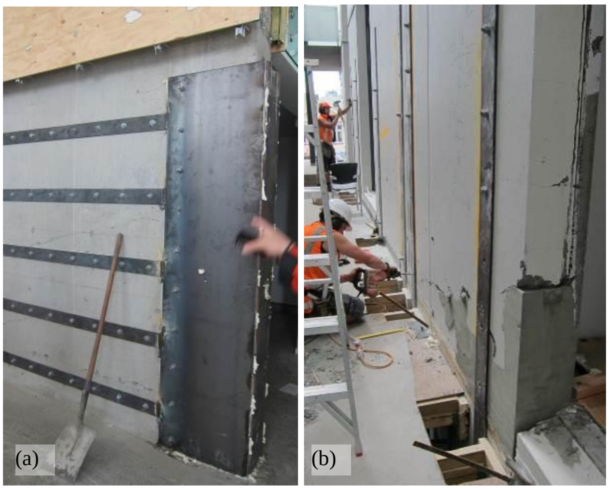

Figure 69: Emergency stabilisation repair of RC walls with a) buckled boundary longitudinal bars with inadequate confinement ties; b) fracture boundary longitudinal bars across a single cracking line.

In Figure 69b, steel plates were added to the wall with fractured boundary bars in order to re-establish the flexural capacity of the wall. As there was inadequate time to assess the extent of the damage properly, the design of the repair work has made certain assumptions that many of the longitudinal bars may have fractured or yielded significantly.

Figure 70: Crack epoxy grout injection repair for RC wall.

Figure 70 above and Figure 29b illustrate the use of epoxy grout injection as a repair method for RC frame and wall elements with a ductile damage mechanism. While an epoxy grout may work to reinstate the concrete compression capacity, reseal the cracks for durability and may improve the serviceability stiffness, it is arguably less ineffective in restoring any concrete-to-reinforcement bond or enhancing flexural and in particular shear capacity under a similar earthquake event.

More experimental work is required to confirm the reliability of standard repairing techniques for different failure mechanisms.

9 PRELIMINARY LESSONS

The 22 February (Lyttleton) earthquake event has, in its complexity, emphasised to the extreme a combination of "old" (either well known or expected to be known) and "new" (not-really expected) lessons possibly in the whole area of earthquake engineering.

It is of interest to note that the SESOC preliminary report on the observations from the Christchurch Earthquakes [72] has made some interim recommendations based on the lessons learnt, some of which are consistent herein.

9.1 Aftershocks effects and design level earthquake

One of the most important lessons is the confirmation of severe misunderstanding between public and scientists, on one hand, as well as potential miscommunication between seismologists and engineers, on the other, on the definition and thus likely impacts of "aftershock" and "design level earthquake".

To a certain extent this is often associated with the use in communication of earthquake magnitude (related to the energy released) instead of shaking intensity (e.g. Modified Mercalli Intensity or ground acceleration)to express the severity of the earthquake.

As shown by the Canterbury sequence, the "aftershock" event on 22 Feb 2011 caused a much more significant "shaking intensity" in the CBD, expressed by the combination of peak ground accelerations, displacement, velocity, duration and energy content visible through response spectra, than the main shock in 4 September 2011.

The general perceptions supported by lack of clear internal or external communication on the matter around the world has typically and, in the wake of Christchurch earthquake, inappropriately suggested that aftershocks following the main event would be "less strong" and thus "less damaging".

The consequence of what could appear to be a simple discussion on semantic and definitions has in fact an extremely important impact on decision making processes particularly when dealing with insurance companies, re-occupation of lightly damaged buildings, and also with repairing/retrofitting and reconstruction considerations.

The complex question to answer is: should current international procedure for building inspection and, to a more detailed and robust degree, detailed seismic assessment of the vulnerability of a building account for the possibility of aftershocks being more damaging than main shock? Also how long such a window of potentially higher aftershocks be kept open (months, years)?

Clearly this would depend on the peculiar characteristics of the local seismicity, but once again, information and better understanding of that can often and apparently be gained with confidence only after the occurrence of a substantial sequence of earthquakes.

More importantly, a clear communication between seismologists and engineers (both structural and geotechnical) and the general public of the technical definitions of the "design level earthquake" and "aftershock" are made.

9.2 Earthquake codes and seismic design – ductile design, MCE, and displacement-based design?

"Earthquakes do not read the code" would be one of the most famous quotes of late Prof. Tom Paulay. However obvious this statement may appear, the actual impacts on the daily practice tend to be forgotten or over-looked.

As a corollary of such a statement, there is nothing such as a spectrum-compatible earthquake event. Design spectrum used in code as well as, in more general terms, all code-requirements, should be used for what they are and meant to be: minimum standards by law, not a target, as it too often are treated. A proper design should thus account for and deal with such uncertainties in a practical and transparent manner.

The capacity design philosophy and ductile detailing are meant to account for the unexpected and uncertainties within the seismic design load level. In some cases however, the use higher elastic strength for a 'nominal ductile' loading within NZS1170:2004 (μ=1.25 and Sp factor=0.925) may give a false sense of robustness based on an elastic force-based design without a verification of the building collapse mechanism.

SESOC interim report [72] has recommended the use of Maximum Considered Earthquake (MCE) as a design limit state in NZS1170:2004. However, if the uncertainty in relation to defining the 'exact' earthquake hazard and loading is considered, perhaps the emphasis should be on a compulsory ductile mechanism design in all seismic loading scenarios for buildings with significant consequences of collapse.

There is a need for a stronger emphasis on a ductile inelastic mechanism (irrespective of the loading), robust load path (with alternative redundant load paths) and good detailing to allow for redundant load-path or "safety-plan" mechanism to be activated should the intended lateral load resisting system not perform as intended.

Furthermore, there is an opportunity and need to recalibrate the current force-based seismic design practice to a more rational and performance-based displacement-design approach (e.g. [65]). The Christchurch earthquake has again demonstrated the need for displacement capacity and compatibility for the entire structure. Within the displacement-design framework [65], structural designers are "forced" into considering the ductile inelastic mechanism, available ductility/displacement capacity (not an arbitrary selected ductility), and the displacement response of the building (instead of displacement response computed by elastic models multiplied by the arbitrary ductile value).

9.3 The impact of acceptable damage to modern buildings and the wider city impact

In general, a large majority of the RC buildings, particularly the modern (post-1976) buildings with capacity-design consideration, performed as expected of them in a severe earthquake, with formation of plastic hinges in the beams, coupling beams and base of walls and columns (e.g. Figure 19 and Figure 28).

As discussed in the preceding Section, a cost-efficient reliable repairing (and strengthening) solution can be particularly complex and delicate design decision. Furthermore, there is a general lack of robust information on procedures and techniques to estimate with confidence the residual ductility capacity of such damaged plastic-hinges in the event of one or more severe aftershock.

As a result of the actual damage and the perceivably excessive uncertainties on the expected performance of the structure in a likely-to-happen moderate-strong aftershock, many of these "modern" multi-storey buildings will be demolished.

Notably, the latter "simple' operation of demolition itself can involve, when dealing with multi-storey buildings, quite an extensive time and not negligible costs.

More importantly, all the above required operations, namely the emergency inspection of building (e.g. BSE Level 1 and Level 2) in the emergency-recovery situation, detailed assessment of the structural damage and expected performance, the design of a repair/strengthening or demolition plan, the council approval and actual implementation of these plans, are inevitably delaying significantly not only the "heavy" reconstruction process, but also, on a daily basis, the accessibility of the CBD area, thus affecting the business operation (downtime) of many close-by buildings

Such considerations on the wider impact of the low performance of a single building to the adjacent or close-by buildings is typically not accounted for when considering retrofit strategies, insurance premium, building consent, etc. A wider vision and plan, looking more at urban scale or at least at a sub-urban area scale, should be adopted in the near future.

9.4 Revisiting Performance-based Design Criteria and Objectives: the need to raise the bar

The excessive socio-economic impact of the 22 February 2011 (Lyttelton) earthquake have confirmed the need to revisit the overall targets set in the current seismic design approach. Similarly, it has emphasised the crucial need for improved communication to the general public and building owners as to what would be the expected level of damage in a code-compliant newly designed or recently strengthened building.

The intention of modern seismic design, or the more recent performance-based seismic design (e.g. the SEAOC Vision 2000 [71]) is generally to minimise life-safety risk on a specific 'design-level' earthquake – typically a 500-year return period earthquake or 10% probability of occurrence in 50 years building life for a normal-use structure. For a rare earthquake (typically a 2,500-year return period earthquake or 2% probability of occurrence in 50 years building life, the collapse risk is minimised.

Figure 71 illustrates these concepts in a performance design objective matrix, which simply indicates the higher the earthquake intensity, the higher the level of damage that should be expected and thus somehow "accepted" (if minimum standards have been adopted).

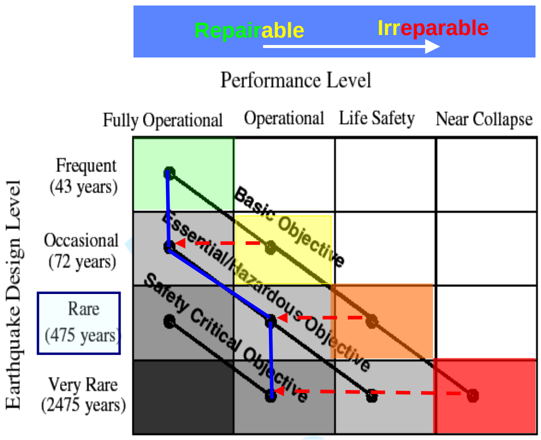

Figure 71: Performance-Based design Objective matrix and modification (blue line) to increase the targeted performance toward a Damage-Control level.

As discussed in other sections, in order to achieve the design objective, the current seismic design of ductile structural systems limits the structural damage to selected discrete "ductile" zone of the structure. However, it inherently implies

that damage, repair cost and building downtime are expected and accepted as unavoidable at the building 'design level earthquake'.

In retrospective, considering the shaking intensity of the 22 February earthquake, in which the seismic loading was at least twice the design level (for a normal-use building), the damage observed to the older and modern buildings was not at all unexpected (for structural engineers).

However but not surprisingly, following the actual impact of a severe earthquake as in 22 February building owners, tenants, insurers, territorial authorities, and public opinion, have a remarkably different expectation of an "earthquake-resistant" building.

As a further confirmation of this lack of understanding and proper communication between technical and non-technical parties involved, the level of damage referred to in the matrix above is mostly associated with the structural part, or the skeleton, with the declared expectations and acceptance that most of the non-structural elements such as partitions, claddings, glazing can potentially be heavily damaged.

Our experience in the September earthquake have shown that, even when the structural skeleton is relatively sound, the direct repairing costs of non-structural elements and the associated indirect costs due to the downtime and business interruption can represent a major component of the overall "losses".

In order to resolve this major perception gap and dangerous misunderstanding, a twofold approach is required [55]:

1. On one hand, it is necessary to significantly improve the communication to the client, insurance, local authorities, and general public, of the seismic risk and expected building performance levels for a given code-compliant design. It must be clear that the targeted performance levels are considered "minimum standards", with the possibility of achieving better performance if desired.
2. On the other hand, it is also possible to "raise the bar" by modifying the New Zealand Building Code, to shift the targeted performance levels from the typically accepted collapse prevention objective under a severe earthquake, to a fully operational objective (with expected capital cost premium to the society). This is represented in Figure 71 by a tangible shift of the objective lines to the left. This will require a societal debate of the acceptable performance and regulatory move towards higher performance levels (or lower acceptable damage levels).

In order to "raise the bar" two clear solutions are available:

- Increase the level of seismic design loading (e.g., increase the seismic coefficient or Hazard Factor Z).
- Move to higher-performance building technology.

As an interim measure for the elevated seismic of the Canterbury region, the design seismic Hazard Factor Z has been raised from 0.22 to 0.3 [16]. Similarly, the requirement for serviceability limit state earthquake (via R factor) has been increased from 0.25 to 0.33.

9.5 Inadequate displacement capacity of secondary or gravity-only elements

The overall and complex implications of displacement incompatibility between the main lateral load resisting systems (or primary elements) and gravity-only or mainly bearing systems (or secondary elements) have been fully recognized in code-design provisions (since the 1994 Northridge earthquake) and yet much needs to be done even in the design of new structures to account for the actual 3D response of the building and required "compatible" movement of its parts.

As already noted in the reconnaissance report from the 4 September earthquake [35], as well as demonstrated in laboratory tests [8], gravity-columns belonging to interior (or exterior) frames designed prior to the 1995 New Zealand Concrete Standards (NZS3101: 1995) may have inadequate displacement/ductility capacity (in terms of transverse reinforcement and confinement detailing). These columns, under moderate drift demand can undergo severe shear damage and thus lose their vertical load carrying capacity.

'Gravity-only' or 'secondary' elements have been observed to either participate as part of the lateral load structural system or displace along the main seismic system. In either scenario, damage, in particular gravity columns and gravity reinforced concrete block walls have been observed. Higher level of displacement demand imposed on these inadequately detailed "secondary" elements can result, as it was observed in few cases, resulting in severe if not catastrophic consequences.

Considering that building displacement response is typically estimated by elastic analysis in the structural design, more emphasis should be placed on adequate detailing of these secondary predominantly-gravity load bearing elements to avoid collapse under a MCE displacement demand.

Similarly, when designing new structures, higher level of redundancy, as discussed in previous paragraphs**,** should be built in, to allow for alternative load path as well as to avoid disproportionate collapse as a consequence of a higher-than-expected event.

9.6 Stairs

As described in Section 7, the collapses and significant damage of stairs in a number of mid- to high-rise modern buildings have raised a serious concern at an international level. Flexible multi-storey buildings with scissor stair configuration with a limited sliding gap detail appear to be the most critical case.

DBH Practice Advisory 13 [17] has outlined some interim measures for assessment and retrofit of stairs in multi-storey buildings in order to avoid the catastrophic collapses observed in the 22 February earthquake.

From the structural perspective, the damage observed in stairs relates to the lack of displacement capacity of its supports and connections. However, considering the crucial role staircases have in terms of safety egress from buildings, re-consideration of the design of staircases is required.

Current design approaches for the design of stairs for adequate displacement demand are available (e.g. [11]). However, considering the difficulty in estimating inter-storey drift using an elastic analysis, higher-than-expected displacement demand should be considered.

Alternative design option such as sliding support on floor slab with conservative seating length (instead of gap-and-ledge arrangement), isolated self-contained stairwell tower (within isolated shear walls) or staircase with redundant catch restraint (e.g. hanger or tie-back detail) or partial-height catch frame/beam (to avoid progressive collapse due to one flight failure).

9.7 Pre-1970s RC buildings vulnerability – time for an active retrofit programme?

Whilst the excessive damage to modern (post-1976) buildings might have come as a partial surprise, partly justified by the high intensity of the shaking, the seismic vulnerability of pre-1970s RC buildings has been internationally well recognized in the last two decades.

In addition to lessons from past overseas earthquakes, and recent research on the seismic vulnerability of RC buildings designed to NZ construction practice (e.g. [66] under FRST-funded Retrofit Solution project), the observed damage of the pre-1970 RC buildings as discussed in Section 4, confirmed the widespread common problems of pre-modern seismic design.

The common list of structural deficiencies of pre-1970s RC buildings in the literature was mostly observed in the 22 February 2011 earthquake. The inherent brittle behaviour of these buildings can tend to a "switch on-off" mechanism, in which elastic response at low levels of shaking may give a false sense of confidence and a brittle collapse may occur in a higher level of seismic shaking.

A paper by the first two authors [59] after the 7.1 M_w 4 September earthquake has highlighted the possibility of severe damage/collapse of pre-1970s RC buildings in earthquake with different shaking characteristics (near-fault motion with directivity or long duration long-period Alpine-fault type motion).

Solutions for strengthening and upgrading existing RC buildings have been developed worldwide and are available. However, similarly to all other countries, in spite of the high risk of collapse of such buildings under a moderate-severe earthquake, there is a lack of enforcement of strengthening/retrofit/seismic upgrading. This is mainly due to the perceived excessive cost to the community and the poor communication of the actual cost-benefit of safer buildings to the community.

Territorial authorities in New Zealand generally have a seismic vulnerability assessment/screening and strengthening of earthquake-prone buildings policy as required by the 2004 Building Act. However, most territorial authorities have a passive policy of which the earthquake-prone buildings policy will only be triggered by a change of use or significant alteration work.

The aftermath of the Christchurch earthquakes have witnessed a rise in public awareness and building owners actions of the seismic vulnerability of these older non-ductile buildings. Therefore, there is a window of opportunity for the seismic engineering industry and local territorial authorities to pursue a more aggressive approach to minimise the seismic vulnerability of these building stock in New Zealand.

If the 1931 Hawkes Bay earthquake has effectively stopped the unreinforced masonry construction practice and raised the awareness of seismic strengthening, 2011 Christchurch earthquake should have the similar effect on removing earthquake prone buildings from New Zealand cities, in particular the pre-1931 unreinforced masonry buildings and pre-1970s RC buildings, either by seismic strengthening or complete demolition of such buildings.

9.8 Irregularity effects (plan and vertical) – inelastic design verification?

In general, buildings with significant plan and/or vertical irregularity were found to perform very poorly. The damage observations presented in previous Sections has highlighted irregularity as one of the main contributing factor in triggering unexpected structural response.

For example, RC walls that discontinued above the basement level were observed to induce severe damage on the transfer slabs and on the basement columns and walls (e.g. Figure 32). Plan irregularity as a consequence of inelastic behaviour of perimeter lateral-resisting systems (walls or frames) leading to inelastic torsion amplification was another observed phenomena.

The irregularity arising from a localised inelastic mechanism of a regular building (e.g. transverse frames yielding prior to the walls in the longitudinal directions etc.) is a complex design problem.

The current seismic Loading Standards NZS1170.5:2004 [41] has a reasonably robust definition of irregularity which will trigger various analysis and design requirements. However, the current practice of reliance on a **3D elastic structural model** to provide demand amplification for an expected inelastic torsional behaviour can be misleading and might not yield the desirable building performance.

Arguably, a simple inelastic analysis such as that proposed by Prof Paulay [62] may yield a predictable performance level, rather than reliance on elastic 3D model. Alternatively, for complex and important structures (e.g. Importance Level 3/4) perhaps the use of non-linear 3D model for seismic design verification is warranted.

9.9 Vertical acceleration, bi-directional loading and variation of axial loading

The vertical acceleration observed in the 22 February 2011 earthquake was very high but is comparable to other near-fault records observed. However, the impact of vertical accelerations on building performance is unclear.

For example, the current design of columns and walls can rely significantly on the vertical axial-load component for their shear and flexural capacities. Whether the excessive vertical acceleration diminishes these vertical axial load is unclear.

Similarly, the design of vertical load-bearing elements (e.g. columns, walls, joints, cantilevered beams and transfer beams) is based on some particular assumptions of the axial loading. The vertical acceleration on columns and walls can result in the variation of axial load and increases the compressive strain demand. In additional, bi-directional loading can also increase and decrease the axial load demand on the columns, walls and beam-column joints,

Conventionally, the variation of axial load is only considered from the lateral actions of the building and not the reduction (or amplification) of the gravity-load components due to vertical acceleration. While it is argued that the vertical acceleration duration is very short and therefore unable to generate sufficient variation of axial load, the high number of compressive-failure of flexural-shear elements may suggest that the design analysis may need to include the variation of axial loads from all possible loading combinations.

9.10 Shear wall detailing and design for confinement and compression

Perhaps some of the most important lessons for modern construction relate to the performance of reinforced concrete wall buildings. While capacity design approaches protected shear walls against shear failures in modern wall buildings, unexpected flexural compression and tension failures in numerous shear walls in Christchurch indicate the need to modify shear wall design provisions to improve the flexural ductility of slender walls. In particular, the following issues deserve further research and should be addressed in future building codes:

- Shear walls designed for nominal ductility, without sufficient boundary zone confinement, can experience brittle concrete crushing in the compression zone. The concrete strain capacity of thin walls without confinement may be less than typically assumed values. Similar observations were made in 2010 Chile earthquake.

- If crushing is avoided through the confinement of the compression zone, shear walls with thin webs unsupported by an enlarged boundary or flange may be vulnerable to buckling of the compression zone. This may be a particular concern for T-, L-, or V-shaped walls where the web can be subjected to high tensile strains followed by high compression strains prior to yielding of the flange reinforcement. Buckling of a wall's web was observed in a well-confined compression zone with storey height to thickness ratio less than 10.
- To avoid brittle compression failures and web buckling, codes may need to limit the depth of the compression stress block to ensure a tension-controlled failure mode can develop in a slender shear wall.
- Fracture of small boundary zone bars in two modern buildings suggests that minimum reinforcement provisions for boundary zones of shear walls should be reviewed.
- The effect of a 'high' concrete tensile strength in inducing high strain demands on the wall reinforcing needs to be quantified via further research.
- Wall design typically assumes a plastic hinge extending approximately half the wall length from the ground level. Damage from the Christchurch earthquake suggests that the hinge may occur above the ground level (potentially outside the confined zone) over a length that is considerably shorter than half the wall length.
- The lack of confinement ties in the web and core of the walls in the plastic hinge region under significant gravity axial load is another area that requires further research.

9.11 Near-fault pulse-like seismic loading

A large number of seismic acceleration records of the 4 September 2010 and 22 February 2011 earthquakes have shown the strong ground motions with forward directivity effects within 20km from the fault. Preliminary analysis of the strong ground motions has confirmed the high velocity pulse and forward directivity effects observed in the CBD records in the 22 February 2011 event [9].

Since the 1971 San Francisco earthquake, the peculiar structural response to near-fault ground motions has been documented [7, 75]. The amplification of seismic wave in the direction of rupture due to forward directivity effect leads to a low-cycles motion with a coherent long period velocity pulse termed as "fling effect". Near-fault motion has shown to cause significant strength, displacement and ductility demand in structures as well as variation in inter-storey shear demand for both long and short period structures [3, 29, 36, 37].

More urgently, modern structures in near fault regions might have inadequate displacement or ductility capacities because near-fault effects are often overlooked or underestimated in design codes. In the NZS1170:5 (2004), the near-fault amplification factor for elastic design spectra was based on a near-fault attenuation model that has been shown to be inconsistent when compared to recorded near-fault ground motion data [74].

McVerry *et al*. [40] cited the lack of near-source records in New Zealand strong-motion database for the lack of a calibrated attenuation model for spectra generation. A preliminary magnitude-dependent response spectra model that is significantly different from existing models used in codes has also been recently proposed [74]. It is expected that further research and analysis of the Canterbury earthquakes seismic records will lead to future revision of the NZS1170.5 to better account for near-fault effects.

A limited number of experimental tests of RC structures under near-fault high-velocity low-cycles excitation are available [12, 64, 67]. These tests generally show a higher transient and residual displacement demands on the RC elements. Strain concentration and concentration of damage was also observed. However, there are inadequate test results to verify or confirm whether some of the observed strain concentration, concentrated flexural cracking, and reduced strain penetration lengths in the 22 February earthquake are consequences of near-fault excitation.

9.12 Soil- Structure Interaction: Integrated design approach to avoid liquefaction induced differential settlement and tilting

In the 4 September 2010 M_w 7.1 Darfield earthquake as well as the 22 Feb 2011 M_w 6.2 Christchurch earthquake, severe widespread liquefaction and lateral spreading were observed in the Christchurch and surrounding suburbs. However, limited or partial liquefaction manifestation was observed within the Christchurch CBD in the 4 September event, while severe liquefaction was observed in parts of the Christchurch CBD in the 22 February earthquake.

The severe widespread liquefaction and lateral spreading observed in the CBD area following the 22 February event, and to a greater extent in many other suburban areas, has led to significant lateral movement or differential settlement in the building foundation systems, resulting in foundation damage and permanent tilting of the structures [26], as shown in Figure 72. Variable soil profiles underneath these buildings with varying foundation designs are some of the complexities resulting in mixed (good and bad) performance of various CBD buildings within the same segment of liquefaction-damaged street.

***Figure 72:** Liquefaction induced differential settlement resulting in significant tilting of mid- to high-rise buildings of various foundation and soil details: a) Four-storey with shallow foundation; b) Six-storey with shallow foundation; c) Two high-rise buildings with substantial differential settlement and tilting.*

Preliminary observations indicate buildings with piled foundations generally exhibit less differential settlement and liquefaction-induced tilt [26]. High-rise multi-storey buildings founded on shallow foundations with significant liquefiable soil depth generally exhibited substantial settlements and liquefaction-induced tilt.

In general, the relative extent of damage and repair/remediation costs associated to the superstructure and to the foundation systems varied significantly. The overall result is that the combination (sum of) this damage and repair costs is ultimately leading many buildings to be demolished.

Although soil-structure-interaction, SSI, or soil structure-foundation-interaction, SSFI, has been recognised for decades as a major and very challenging topic in earthquake engineering, much more effort is needed to develop and provide user-friendly and practical guidelines for the practitioner engineers.

Performance-based seismic design as described in Section 9.4 can be extended to include combined performance criteria and acceptable limit states for the superstructure and foundation-soil structure.

9.13 Brittle mesh and beam-elongation effects on precast floor diaphragm

Diaphragm action is a complicated issue as the induced forces in the diaphragm can be very high due to the in-plane stiffness of the floor and the induced diaphragm forces from beam elongation and slab-flange actions.

The vulnerability of cold-worked wire mesh for diaphragm action has been recognised since the mid-2000s, as per the DBH Practice Advisory 3 [15] and the Amendments No. 3 to NZS3101 in 2004 [47]. The reliance on cold-drawn wire mesh for the inertial force transfer between the diaphragm and the main lateral-load resisting elements can be very un-conservative as the required strain can be significantly higher than expected.

As discussed in Section 5.1, the displacement-incompatibility of lateral load resisting systems and the precast floor diaphragm, arising from the adverse elongation effect of expected ductile plastic behaviour of RC frames [20, 39, 47], in conjunction with the use of brittle mesh for topping reinforcing can lead to a very vulnerable outcome (as observed in the building in Figure 30).

It is noted the duration and number of inelastic cycle demands in the 22 February earthquake is short and limited. A longer duration severe earthquake can potentially lead to more severe diaphragm failure and perhaps collapse of the floors shown in Figure 30.

As noted in the SESOC report [72], there is a need for simple and unified design guidelines for diaphragms, irrespective of the material of the primary structural elements. While the current practice of using either earthquake-grade "ductile" mesh reinforcing or using ductile mild steel reinforcement for shear transfer from the diaphragm appears to be performing satisfactorily in inspected buildings, the need of thorough intrusive inspection of the damaged floors can render the building to be uneconomical to repair.

Alternative design solutions for precast floor diaphragm transfer such as mechanical shear key on un-topped floors (e.g. USA practice [76]) or un-bonded long tie-back reinforcements can be considered and researched for future application.

9.14 IEP Assessment

The Initial Evaluation Procedure (IEP) Assessment following the 2006 NZSEE Guidelines [52] is a widely used seismic assessment screening tool in New Zealand. While the IEP assessment is an economical and rational framework to screen for Earthquake-Prone Buildings (EPB), the 22 February earthquake has also highlighted some of its limitations

The IEP assessment and the 2006 NZSEE Guidelines [52] have popularised the concept of *Percentage of New Building Standards* (*%NBS*) as a measure of seismic vulnerability of buildings. However, the level of 33*%NBS* used in regulations to the Building Act 2004 to define an earthquake-prone building has been wrongly interpreted as meaning that buildings above this level are relatively safe in a major earthquake. This is in spite of clear messages to the contrary that the legislation was set to cover only the worst of buildings. The %NBS score is further misleading if it is derived from a very crude IEP assessment (e.g. without any structural drawings or site inspection).

The IEP assessment have highlighted four critical structural weaknesses such as plan and vertical irregularity, short columns and pounding potential, with each having a similar weighted reduction factors (Factors A to D). Some of these factors are valid indicators of poor structural performance for RC buildings, as evidence of the various structural failures which arise from say plan and vertical irregularity discussed in the preceding Sections.

Some factors (e.g. short columns and pounding) are typically more relevant to certain typology of buildings such as unreinforced masonry buildings. There is little evidence in Christchurch which suggests significant damage or structural failure of RC buildings due to seismic pounding for example. Figure 73 illustrates some of the more common 'localised' damage as a consequence of pounding. However, it is noted that experience from overseas earthquakes have shown the severe effects of seismic pounding for RC buildings (e.g. in Mexico City 1985 earthquake [68]).

Figure 73: Pounding damage was not widely observed.

However, the use of equal-weighting and a limited list of 'critical structural weaknesses' tend to draw attentions away from some other issues that may lead to catastrophic collapse and loss of lives. Various critical structural weaknesses as

highlighted in this report – such as brittle mesh diaphragm reinforcing or a poor diaphragm-to-lateral load resisting system, non-ductile pre-1970s RC building detailing, gravity columns, non-ductile walls etc. are generally not explicitly considered in such assessments.

The IEP assessment, which essentially is a screening and rapid assessment tool, is increasingly used as the 'standard' entry-level seismic assessment of existing buildings. It should be highlighted that the IEP assessment alone is unlikely to be able to capture most of the RC buildings with fatalities in the 22 February earthquake.

9.15 Structural drawings repository for emergency structural assessment

The availability of construction drawing of particular classes of buildings that are identified as highly vulnerable or significant (e.g. higher than 6-storeys) can be very useful to the search and rescue efforts. In New Zealand, various local territorial authorities have varying policies and timeframes in digitising the council records (and building drawings). The management of a large volume of data/information that is urgently needed in the event of emergency can present challenges in establishing building inventory (and drawings repository). Such repository within the local territorial authorities should be considered as a critical emergency resources and high priority.

10 FINAL REMARKS

This paper has presented a summary and overview of preliminary lessons from our observations of the seismic performance of RC buildings in the 22 February 2011 Christchurch earthquake.

Due to the concise nature of the paper and relative to the amount of information collected and observed, it was not possible to discuss all relevant aspects in details. At the time of writing, the Royal Commission of Inquiry and various investigations on the seismic performance of severely damaged and collapsed RC buildings are on-going. Readers are encouraged to read the outcomes of the inquiry at the Royal Commission website [70].

An observational damage report comprising more than 100 RC Buildings has also been compiled [57] as part of the Natural Hazard Platform Recovery Projects.

The unique and unprecedented series of severe earthquake events in Christchurch and the substantial damage observed to the older "non-ductile" and also modern and "well" designed RC buildings is an invaluable 'learning lesson' for earthquake engineering. It is essential that comprehensive efforts are undertaken to further analyse and study the lessons from these earthquakes.

The Canterbury earthquakes have also started the discussion for improvement in the building design and the underlying performance objectives that will fulfil the expectation of New Zealand society of its built environment.

As with previous major earthquakes around the world, the Christchurch earthquakes provide a window of opportunity for the New Zealand construction industry to recognise and deal with some for the existing vulnerabilities, as well as, to pursue a more aggressive approach to minimise the seismic risk of the building stock in New Zealand.

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12 DISCLAIMER

Any opinions, findings and conclusions or recommendations expressed on this report are those of the author(s) and do not necessarily reflect the views of any associated organisations or entities. While this report is factual in its nature, any conclusion and inappropriate mistake in reporting made in this contribution are nevertheless to be considered wholly of the authors.

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