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22 FEBRUARY 2011 CHRISTCHURCH EARTHQUAKE AND IMPLICATIONS FOR THE DESIGN OF CONCRETE STRUCTURES

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ABSTRACT: At 12:51pm local time on 22 February 2011, a Mw 6.2 aftershock of the September 4, 2010, Darfield Earthquake shook the city of Christchurch, New Zealand. The aftershock occurred on an unmapped fault less than 8 km from the city center resulting in the collapse of two reinforced concrete office buildings and one concrete parking garage, and severe damage to numerous others. This paper summarizes the observed damage to concrete buildings in Central Business District (CBD), with a specific focus on identifying future research to support possible changes to international seismic design codes to address the Christchurch observations.

Key Words: Christchurch earthquake, ground motions, concrete buildings, collapse, code changes, shear walls, precast moment frames, repair costs.

INTRODUCTION

Six months after the 4 September 2010 M_w 7.1 Darfield (Canterbury) earthquake, the M_w 6.2 Christchurch earthquake struck Christchurch, New Zealand on the 22 February 2011. The M_w 6.2 earthquake occurred on a previously unknown fault less than 10km south-east of the Christchurch central business district (CBD), initiating at a shallow depth of 5km. Unlike the 4 Sept event, when limited-to-moderate damage was observed in engineered reinforced concrete (RC) buildings (Kam et al. 2010), 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 highlights important observations of damage to RC buildings in the 22 February 2011 Christchurch earthquake. Focus will be on damage to modern concrete frame and wall building and the identification of future research directions necessary to develop appropriate concrete code provisions to address the observations in Christchurch. Detailed description of damage to concrete buildings of all vintages and types can be found in Kam et al. (2011).

GROUND MOTIONS

Recorded ground shaking in the CBD matched or exceeded the 2500 year return period design spectrum from the NZ Standard (NZS1170.5 2004), particularly in the east-west direction (Figure 1). Using NZS 1170.5, normal buildings are designed for the 500 year return period design spectrum using ultimate limit states design principles, while post-disaster buildings are designed using the 2500 year design spectrum shown in Figure 1.

From disaggregation of the seismic hazard, a M_w 6.2 earthquake at a distance of 10 km did not contribute significantly to the probabilistic seismic hazard model used for Christchurch (Stirling et al 2002). With this in mind it raises the question if considering relatively large area sources in probabilistic seismic hazard assessment to account for earthquakes on unknown faults is sufficient to capture the risk unknown faults may pose to our urban regions. Should we consider deterministic scenarios (i.e., shaking from a specific magnitude at a specific distance) for seismically active regions where unknown faults may dominate the earthquake risk? Research is required on the most appropriate means of accounting for seismic hazard posed by earthquakes on unknown faults.

One unique aspect of the 22 February earthquake was the very high vertical ground accelerations, frequently exceeding the peak horizontal accelerations at recording station within approximately 10 km of the epicentre. Similar to past earthquakes, the vertical accelerations were characterised by very high frequency content and peak values were only attained for a very short duration. Research is needed to define the importance of vertical excitations on building performance considering the high frequency content and phasing with lateral demands. Rapid attenuation of high frequency vertical ground motion should also be considered as this may only be critical for near-fault events.

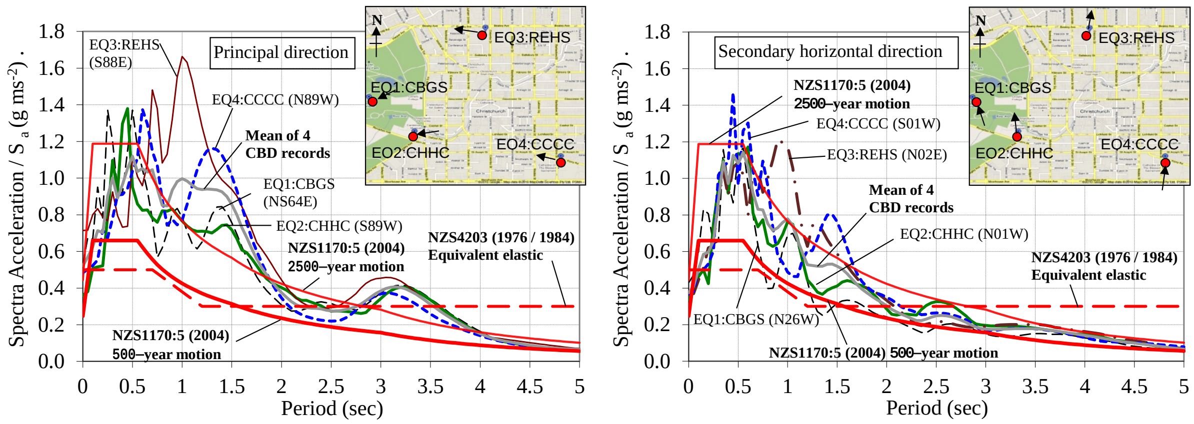

Fig. 1 Elastic horizontal acceleration response spectra (5%-damped) from Christchurch CBD and the NZS1170.5 design spectra (red) for Christchurch (NZS soil class D, R=20km)

CONCRETE BUILDING TYPES AND DAMAGE STATISTICS

With a population of approximately 390,000, Christchurch is the second largest city in New Zealand and the economic centre of the South Island. As a relatively centralized city, approximately 25% of the total employment in the city was located in the Christchurch CBD, leading to a concentration of buildings over five storeys in the city centre. In the CBD there were 127 buildings with at least six stories, with the tallest RC building being 22-storey (86 metres). RC frames and RC walls are the most common multi-storey construction types. Out of 183 buildings with more than 5-storeys, 49% are RC frame buildings, 22% are RC wall buildings, 7.7% are reinforced concrete masonry and 5.5% are RC frame with infills. Only 9 steel structures with more than 5-storeys were observed in the CBD.

Buildings constructed prior to the introduction of modern seismic codes in mid-1970s are still prevalent in the Christchurch CBD. Approximately 45% of the total CBD building stock was built prior to the 1970s. Of this, 13.8% or 188 pre-1970s buildings are with 3-storey and more.

Precast concrete floor systems have been used for multi-storey RC buildings in New Zealand since the mid-1960s. From 1980s to present, the majority of multi-storey RC buildings used precast concrete floors or concrete composite steel deck systems. Ductile precast concrete frames, designed with wet connections to emulate cast-in-place construction, were introduced in early the 1980s and soon became the most popular form of construction for RC frames.

Figure 2 provides statistics for building safety evaluation placarding of concrete frame and wall buildings in the CBD as of June 12, 2011 (i.e. prior to June 13 aftershocks). While not being a refined measure of damage, coloured placarding, following procedures similar to ATC-20 (NZSEE 2009), provides a general indicator of the distribution of damage to building types and vintages. In the interest of brevity only concrete frames and walls are reported here; statics for all building types can be found in Kam et al. (2011). Statistics for frame buildings indicate a relatively consistent level of performance for frames constructed prior to 1990, with a slight improvement in performance after 1990. In contrast, for wall buildings the percentage of red placards was markedly higher for post-1990 buildings compared with buildings constructed prior to 1990. Furthermore, the percentage of green placards remained reasonably consistent for all vintage of wall buildings. The relatively high level of damage observed in concrete wall buildings will be discussed with reference to specific observations in the following sections.

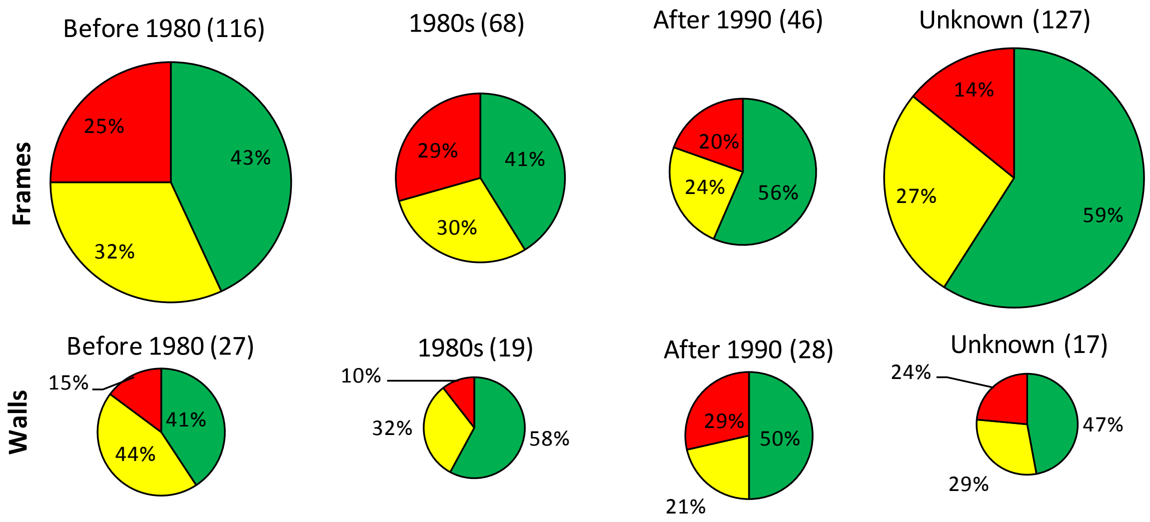

Fig. 2 Distribution of placards for concrete frame and wall buildings in CBD as of 12 June 2011, sorted by age of construction. Total number of buildings in each group given in parentheses above pie chart and approximately reflected in pie chart size. (Data source: Christchurch City Council)

When considering the structural performance implied by the statistics in Figure 2, it is important to recall the level of ground shaking relative to the earthquake demands assumed in design (Figure 1). Normal buildings designed to NZS 1170.5 and the concrete design code (NZS3101 2006) are expected to have a “small margin against collapse” for the 2500 year design spectrum, assumed to be 1.8 times the 500 year design spectrum (King et al. 2003). Considering the high spectral demands (Figure 1) and that the two concrete office buildings that collapsed were designed in 1960s and 1980s, it might be concluded from the point of view of collapse and life safety performance that concrete buildings designed according to recent building codes performed well or as expected during the Christchurch earthquake.

While potentially satisfying the objectives of the code, the level of damage observed in many concrete buildings was severe, leading to a large percentage of buildings currently considered uneconomical to repair. Potentially more than 50% of buildings in the CBD will be demolished as a result of damage from the February earthquake (EERI 2011). Fig. 3 shows beam hinging observed in a 22-storey moment frame building. The damage was consistent with the prevalent capacity-design philosophy, protecting the columns from damage and concentrating nonlinear response in the beam hinges. However, wide residual cracks in the beams have raised concerns regarding low-cycle fatigue of the reinforcement and repairability of the building. Moving forward, the financial risk and damage acceptance of ductile RC systems may require further considerations. It is not clear if the performance of this and other buildings in Christchurch is acceptable to society or if society is willing to pay more for better performance in future earthquakes. However, to enable the selection of different performance levels in the future, the engineering community should use the impacts of the Christchurch earthquake to promote the further development and implementation of performance-based seismic design approaches (e.g. ATC-58 2011).

Fig. 3 Beam cracking in 22-story RC moment frame (precast emulating cast-in-place)

EXAMPLES OF DAMAGE TO CONCRETE WALL BUILDINGS

Perhaps some of the most important lessons for modern construction from the Christchurch earthquake relate to the performance of reinforced concrete wall buildings. Most shear walls in CBD buildings were tall slender walls where, after the 1982 Concrete Code (NZS 3101, 1982), capacity design concepts were applied to ensure flexural yielding at the base of the wall limited the shear demands and sufficient horizontal reinforcement was provided to avoid shear failure in the plastic hinge zone. While this design approach appeared to protect 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. The following provides a brief summary of some examples of failure modes observed after the 22 February earthquake including web buckling, boundary zone and web crushing, and boundary zone steel fracture.

Wall web buckling

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

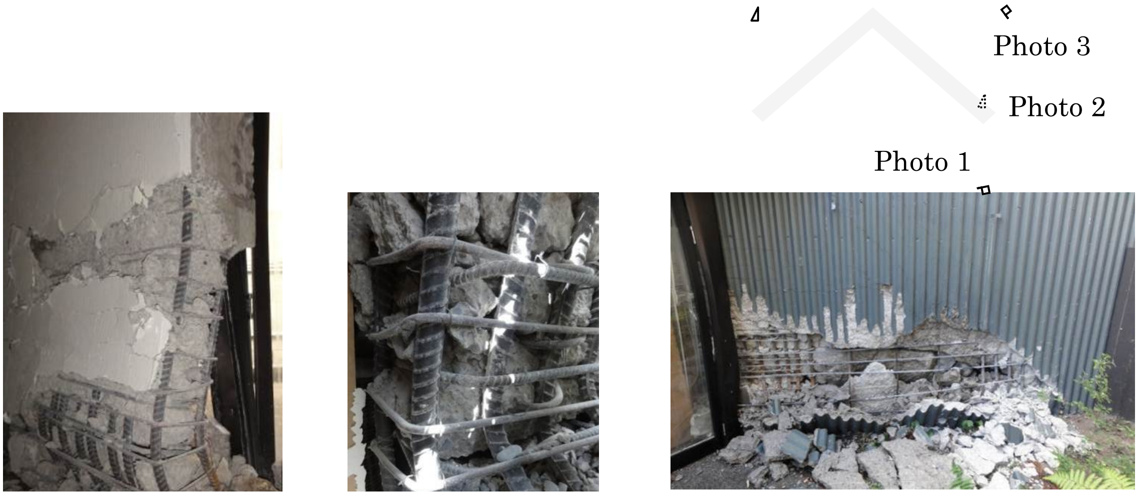

Fig. 4 Seven-storey 1980s office block with compression failure of the V-shaped RC shear wall.

The wall buckled over a height of approximately 1 m and crushing extended over 3 meters 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 story height. Well distributed, primarily horizontal, cracking with widths less than 1.5 mm, were observed in the lower half of the first story of the generally undamaged flange. The damage pattern shown in **Error! Reference source not found.** 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.

Buckling in a wall with a height to thickness ratio of less than 10, a limit used in several international codes avoid out-of-plane instability, suggests that a limit on *h/t* may not be enough to prevent wall buckling. Building on the work by Paulay and Priestley (1993), further research is needed to determine a relationship between wall buckling, length of plastic hinge and axial stiffness of the compression zone after bar yielding.

Boundary bar fracture

Fracture of very light longitudinal reinforcement was also noted in modern high-rise buildings. In some cases (e.g. Figure 5), wide spacing of transverse reinforcement may have led to bar buckling prior to bar fracture. The architectural design of the building in Figure 5 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. In some lightly reinforced shear walls only exhibiting symptoms of flexural cracking, bar fracture of longitudinal bars was detected only after removal of cover concrete. This has significant implications for both design of lightly reinforced walls but also inspection of similar walls after future earthquakes. Engineers performing post-earthquake assessments need to be cautious when assessing the extent of damage to lightly reinforced shear walls.

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

As shown in Photo 1 of Figure 6, 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 (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 was spalled. 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.

In terms of future code development, the concrete crushing within the web of the wall in Figure 6 suggests that cross ties may be required outside the confined end zones and splices should be avoided in transverse bars. Additionally, bar fracture of the lightly reinforced walls shown in Figures 5 and 6 suggests that the minimum reinforcement provisions for boundary zones of shear walls should be reviewed.

Fig. 5 Buckled and fractured bars in lightly reinforced slender RC shear walls.

Fig. 6 Boundary bar fracture and web crushing in modern 8-story apartment building.

Wall crushing

Figure 7 shows severe damage to a shear wall in the 22-story Hotel Grand Chancellor, designed and constructed in mid 1980s. Shortening of the ground-floor wall by approximately 800 mm led to a visible lean of the building and restricted access to the potential fall zone around the building. Significant structural layout irregularities influenced the seismic response of the building; most notably the east side of the building was cantilevered over an access lane. Furthermore, the seismic force resisting system for the lower 14 storeys consisted of shear walls, while perimeter moment frames were used for the upper stories. (Description below focuses on the performance of the damaged wall; further details about the building and the complex damage pattern observed can be found in Dunning Thornton (2011) or Kam et al. (2012).)

Damage shown in Figure 7 indicates that the wall displaced downward 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.

The wall shown in Figure 7 was likely supporting very high axial loads from several sources. First, the wall supported a disproportionately high tributary area since it acted as a prop for the cantilevered bay on the east side of the building. Secondly, the corner column of the upper tower perimeter moment frame would have imparted high axial loads to the wall due to overturning moments, particularly with any bi-directional movement to the south-east. Thirdly, vertical excitation of the cantilever structure could have exacerbated the axial load on the wall. Finally, the structural wall 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.

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 at the base of the wall, likely further contributed to the location of failure observed in Figure 7. Future research is needed on the influence of the out-of-plane movement of shear walls when combined with high axial loads.

Fig. 7 Crushing and out-of-plane movement of shear wall in 22-story Hotel Grand Chancellor (Middle and right photos from Dunning and Thornton (2011))

Observed wall crushing failures, including that shown in Figure 7, suggest that it is best to avoid compression-controlled walls. It should be recognized, however, that the true axial loads on walls are not well known, in part due to growth of a wall during shaking and the outrigger effect from gravity columns. Codified limits on wall axial loads are being considered by the New Zealand Department of Building and Housing in response to the damage observed in Christchurch (DBH, 2011).

DISPLACEMENT COMPATIBILITY

Similar to past earthquake, the Christchurch earthquake demonstrated the need to carefully consider displacement compatibility in the design of concrete buildings. The following sections highlight three specific issues related to displacement compatibility observed in Christchurch; namely, collapse of precast concrete stairs, near-unseating state of precast floors, and severe damage of "gravity system" not detailed for adequate ductility capacity.

Fig. 8 Collapse of precast stairs

Precast Stairs

Precast stair units collapsed in at least four multi-story buildings, and were severely damaged in several other cases, as a result of the 22 February 2011 Christchurch Earthquake. Figure 8 shows collapsed stair units from the 18-storey Forsyth Barr Building (described in detail by Beca (2011)) and the Hotel Grand Chancellor. Stair damage was particularly prevalent in buildings relying on moment-frames for lateral load resistance, although stair damage was also noted in some shear wall buildings. Collapse of stairs not only pose an immediate life safety hazard during the earthquake, but can result in death or injury after the earthquake as occupant attempt to evacuate the building in dark conditions without prior knowledge of the stair collapses. Loss of egress routes as a result of the stair collapses necessitated the evacuation of occupants from windows of high-rise buildings following the Christchurch earthquake.

Stairs are typically designed not to resist building deformations during earthquakes. Movement of the building relative to the stairs is generally accommodated by seismic gaps and seating provided at one end of the stair unit. The seismic gap and seating support must be sized for the expected drift demands during an earthquake. The drift demands from the 22 February earthquake exceeded the 500-year design drift demands required by the New Zealand loading standard (NZS 1170.5, 2004) for most building periods. Review of stair collapses in the Forsyth Barr building (Beca, 2011) indicates that the seismic gap provided was not large enough to avoid closure of the seismic gap and development of compression forces in the precast stair units. Debris or construction imperfections may have further decreased the seismic gap, increasing the likelihood of compression in the stair units. Expected compression loads could have resulted in yielding at the landing and shortening of the stair units (Beca 2011). Upon reversal of drift demands on the building, the shortened stairs were particularly vulnerable to unseating and collapse. Limited seating support would have also increased the likelihood of progressive collapse once collapse was initiated at one story.

Although damage to stairs was noted in some buildings with shear walls, damage observed in Christchurch suggests that buildings with moment frames may be particularly vulnerable to stair collapse. Hinging of beams in moment frames leads to shear distortions in a building frame bay resulting in lengthening of bay diagonal, and hence, more movement at the stair support. Furthermore, hinging in beams can result in beam elongation, if not sufficiently restrained by the diaphragm, leading to further displacement demands at the stair support. Stair collapses in the Clarendon Towers, where precast diaphragms detached from the exterior frames (see Figure 9), have been, in part, attributed to the additional deformation demands from beam elongation (Bull 2011).

The poor performance of stairs in the Christchurch earthquake raises several important considerations for the design and evaluation of buildings internationally. Considering the need to provide egress for occupants of damaged buildings after a major earthquake, it is important that the drift demand used to size the gap and seating be reflective of that expected in the maximum considered earthquake. In light of the uncertainty in the ground motions, differences in the linear and nonlinear displacement profile of the building, and the lack of redundancy when seating support is lost, a detail insensitive to construction tolerances or obstructions and allowing for larger than design displacements should be adopted. Ideally to avoid unintended compression in the stair unit, seismic gaps should be avoided by allowing the stair unit to slide on the top of the slab surface (Beca 2011).

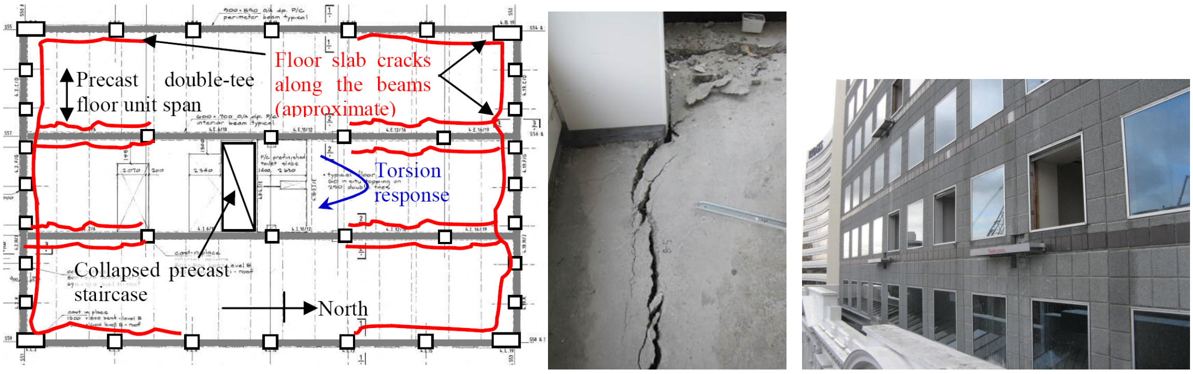

Fig. 9 Separation of precast floor slabs from supporting perimeter moment frame in 18-storey office building. Photo to right (courtesy of D. Bull) shows temporary supports for columns after earthquake.

Precast diaphragm and beam elongation

Figure 9 illustrates an extreme example in which extensive floor diaphragm damage with near loss of precast flooring unit supports was accompanied by beam elongation in a perimeter moment frame. The building shown in Figure 9 is a 17-storey building with ductile perimeter moment frames, internal gravity frames and flange-hung supported precast double-tee flooring. 60mm topping with cold-drawn wire mesh reinforcement is used. The perimeter frames have typical 500x850mm deep precast beams with 600mm square and 800mm square columns. The beam spans are typically 2900mm in the East-West direction and 5800mm to 6500mm in the North-South direction.

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 may have exhibited a moderate level of torsional response, which amplified the demand on the Northern East-West perimeter frame.

Due to the high beam depth-to-span ratio (850/2900), the beam elongation effects (geometrical elongation and plastic cyclic 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 by approximately 10 to 15mm, indicative of loss of precast floor seating support.

While floor collapse did not occur, the separation of perimeter frames from the diaphragms at multiple levels in the Clarendon Towers (Figure 9) raises concerns of possible much greater consequences due to column buckling in aftershocks. To address this risk of collapse in future aftershocks, pairs of 25 mm rods spanning the full width of the building and attached to spreader beams were installed at each vulnerable column at four heavily damaged levels (see Figure 9). Where similar precast floor systems are used, it is important to ensure there is strong connection between the diaphragm and the frames through a well reinforced topping slab in order to ensure separation between the frames and precast components is minimized. Welded-wire mesh should be avoided in topping for precast diaphragms given their limited strain capacity. Only ductile welded-wire mesh has been allowed in New Zealand since 2005 (DBH 2005).

Fig. 10 Damage to gravity columns

Gravity columns

The Christchurch earthquake also reinforced the need to consider deformation compatibility of the gravity system with the seismic force resisting system, particularly for so-called “gravity columns” not assumed to resist lateral loads but essential for gravity load support. Figure 10 shows varying levels of damage to gravity columns in three different buildings. Photo on right shows one of the critical weaknesses of the collapsed CTV Building, 400mm diameter columns with 6mm ties at 250mm (for further details on the CTV building see Kam et al. (2011)).

The gravity system must be able to accommodate the deformations imposed by the seismic force resisting system during strong ground shaking. After the axial load failure of several gravity columns during the 1994 Northridge Earthquake, ACI 318 introduced provisions for “components not part of the lateral force resisting system”. For columns deformation compatibility must be explicitly checked or confinement must be provided to ensure adequate deformation capacity. Similar provisions have been adopted in other international codes; however, continued research is required given the challenges of evaluating both deformation demands and capacity accurately.

CONCLUSIONS

Codes controlling the seismic design of buildings evolve over time, with the greatest advances often accompanying damage observations from severe earthquakes. Observations from Christchurch are expected to ultimately impact codes in New Zealand (DBH 2011) and internationally. This paper has provided examples of observed damage to concrete buildings that relate directly to future research needs and potential future changes for seismic design practice and codes.

Some of the important observations and recommendations are summarized below:

- The recorded ground motions in the Christchurch CBD are approximately equivalent to the 2500 year design spectrum from the NZ Standard, NZS 1170, for many period ranges of interest.
- Despite the excessively strong shaking demand, performance of most modern concrete buildings generally exceeded the life safety objectives of the code.
- While many buildings met code objectives, an increasing number of concrete buildings are now being considered uneconomical to repair, once again raising the question if collapse prevention is the appropriate target performance level for the building code.
- Research is needed to determine the effects of high frequency vertical ground motion on building structures and how this shaking should be accounted for in design.
- Research is needed to better understand the brittle damage and failure mechanisms observed in walls. Typical wall damage indicates confinement may be required in regions of distributed web reinforcement and over a height exceeding the assumed plastic hinge length. Similarly provisions to limit axial load ratio and slenderness ratio should be evaluated.
- In several occasions bar fracture at critical sections was accompanied by limited flexural cracking. Research is needed to understand this failure mode and develop relevant code provisions.
- Displacement compatibility was shown to be a critical issue in (a) collapse of precast concrete stairs, (b) close-to-unseating state of precast floors due to beam elongation in the moment resisting frames, and (c) the severe damage of “gravity columns” not detailed for adequate ductility capacity. Refinement of code provisions is needed to ensure all systems are able to withstand expected displacement demands.

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REFERENCES

ATC-58 (2011), “75% Draft Guidelines for the Seismic Performance Assessment of Buildings: ATC-58-1” Applied Technology Council, Redwood City, California.

Beca (2011). *Investigation into the Collapse of the Forsyth Barr Building Stairs on 22nd February 2011. A report to the Department of Building and Housing (DBH).* Beca Carter Hollings & Ferner Ltd., Auckland, New Zealand.

Bull, D. K. (2011). "Stairs and Access Ramps between Floors in Multi-storey Buildings. A report o the Canterbury Earthquakes Royal Commission.", Holmes Consulting Group, Christchurch, N.Z.

DBH (2005), *Beware of limitations: Cold-worked wire mesh*, Department of Building and Housing (DBH), Wellington, New Zealand.

DBH (2011). *Technical Investigation into the Structural Performance of Buildings in Christchurch*, Department of Building and Housing (DBH), Wellington, New Zealand, September 2011.

Dunning Thornton (2011). *Report on the structural performance of the Hotel Grand Chancellor in the earthquake on 22 February 2011. A report to the Department of Building and Housing (DBH).* Dunning Thornton Consultants Ltd., Wellington, New Zealand.

EERI (2011) “The M 6.3 Christchurch, New Zealand, Earthquake of February 22, 2011,” *EERI Newsletter*, April 2011.

Kam, W. Y., Pampanin, S., Dhakal, R. P., Gavin, H., and Roeder, C. W. (2010). "Seismic performance of reinforced concrete buildings in the September 2010 Darfield (Canterbury) earthquakes." *Bull. of New Zealand Soc. of Earthquake Eng.*, 43(4), 340-350.

Kam, W.Y., Pampanin, S., and Elwood, K. (2011) “Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttelton) earthquake”, Bull. of New Zealand Soc. of Earthquake Eng., Vol. 44, no. 4, Dec 2011.

King, A.B., Bull, D.K., McVerry, G.H., and Jury, R.D. (2003) “The Australia/New Zealand Earthquake Loadings Standard, AS/NZS 1170.4” 2003 Pacific Conference on Earthquake Engineering, 13-15 February 2003, Christchurch, New Zealand: New Zealand Society for Earthquake Engineering, Paper Number 138.

NZS1170 (2004). *NZS 1170:2004 Structural design actions*, Standards New Zealand, Wellington, NZ.

NZS3101 (2006). *NZS 3101:2006 Concrete structures standard*, Standards New Zealand, Wellington, NZ.

NZSEE (2009). *Building safety evaluation during a state of emergency. Guidelines for territorial authorities. 2nd ed.*, New Zealand Soc. for Earthquake Eng. (NZSEE), Wellington, NZ

Paulay, T. and Priestley, M.J.N. (1993). Stability of Ductile Structural Walls, *ACI Structural Journal*, American Concrete Institute, Vol. 90, No. 4.