Australian Earthquake Engineering Society 2011 Conference, 18-20 November, Barossa Valley, South Australia

Lessons from the February 22nd Christchurch Earthquake

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Introduction

This paper gives a summary of a reconnaissance mission conducted by the author on behalf of the AEES after the February 22nd 2011 (Christchurch) earthquake. It highlights the damage observed in reinforced concrete buildings, the effects of liquefaction and also of the large vertical accelerations generated by this near-field event. Some observations made by others are included to give a more complete picture than could be obtained by one person in the space of four days.

The reconnaissance trip commenced on the 9th of March, two weeks and one day, after the Christchurch earthquake. A preliminary report (Goldsworthy, 2011) and a more detailed report (Chouw, Hao and Goldsworthy, 2011) on the key observations made over the four-day trip can be found on the AEES website. On the first two days the author, Professor Hong Hao from U.W.A. and Associate Professor Nawawi Chouw from the University of Auckland observed first-hand the dramatic effects of liquefaction, ground vibrations and landslides on buildings, roads and bridges in the area surrounding the city centre, and also close to the fault rupture at the port city of Lyttelton. Researchers from the Department of Civil and Natural Resources Engineering at the University of Canterbury, Greg Cole, a Ph.D. student, and Associate Professor Rajesh Dhakal, accompanied the group and provided local guidance and advice. The last two days were spent conducting Level II building assessments in the cordoned-off city centre on a voluntary basis for the Christchurch City Council.

The observed damage to buildings was sometimes associated with inappropriate design and construction practices such as structural irregularity in the horizontal and/or vertical directions, lack of continuity, poor anchorage and connectivity of structural components, and lack of separation between adjacent structures. Unreinforced masonry buildings performed poorly, unless they had been comprehensively strengthened. Older reinforced concrete buildings (pre-1980s design) were known to be vulnerable even though they had behaved reasonably well in the September 2010 Darfield earthquake. (Pampanin et al, 2011) refer to the on-off nature of the structural response of these structures, i.e. reasonable behaviour until their ductility is exceeded and then dramatic and sometimes totally catastrophic failure. More recent (from the 1980s onwards) medium-rise and tall buildings had been designed in accordance with capacity design principles. Given that the ground motions due to the Christchurch earthquake generated much higher levels of acceleration and displacement response throughout the frequency range relevant to built structures than the design level earthquake (typically designed for a 500 year return period), it is not surprising that some of these buildings suffered extensive damage in the more recent event. The damage was usually in the regions of the building that had deliberately designed to be weak and to act as "structural fuses" in a large event, dissipating energy in a controlled cyclic manner.

One interesting phenomenon observed in Christchurch that has not commonly been reported after previous earthquakes in other countries is that of damage caused by a strong vertical pulse. Another defining feature of this event was that the Christchurch CBD and suburbs (particularly the eastern suburbs along the Avon river, and also the southern suburbs) are highly vulnerable to liquefaction and this earthquake resulted in the effects of liquefaction being spread extensively through these areas.

There are salutary lessons for Australian engineers to be gained from our observation of the effect of a level 6.3 magnitude shallow earthquake at close range to the CBD of a city. Given that one of the Australian capital cities is likely to experience an earthquake of this magnitude every few thousand years it forces us to reflect on the adequacy of our existing design philosophy.

1 Seismology in the Canterbury region

Even though the city of Christchurch is at a considerable distance from the plate boundary between the Pacific Plate and the Indo-Australian Plate, it is in a seismological region that is influenced by the complex movements along this plate boundary. The secondary faults near to Christchurch result from the bend in the plate boundary to the north of the city. A deep bed of river gravels covering the Canterbury Plains has hidden the evidence of previously active faults in this region and, even though seismologists were aware that such faults existed, both the Darfield and Christchurch earthquakes were generated by faults that had not previously been mapped and identified. The 6.3 Magnitude Christchurch earthquake generated highly damaging ground motions because it occurred at a shallow depth of only 5 km and the rupture zone of approximately 10 km x 10 km was oriented such that it passed directly under the CBD. The location of the earthquake epicentre relative to the Christchurch CBD is given in Figure 1 (NZSEE website, 2011). In this type of event it is expected that ground shaking will be severe within a distance of about 10 to 20 km of the epicentre.

Figure 1. The location of the earthquake epicentre relative to the Christchurch CBD

Strong ground motions from the Christchurch event were recorded at four sites throughout the city and at other locations in the vicinity of Christchurch. The recordings from the Christchurch Cathedral College (CCC) site are given in Figure 2 (NZSEE, 2011). It should also be noted that recorded vertical accelerations were sometimes higher than the largest recorded horizontal accelerations at a measuring station, probably due to the close proximity of the upthrust from the fault. For more information on the seismology of the region refer to (GNS, 2011).

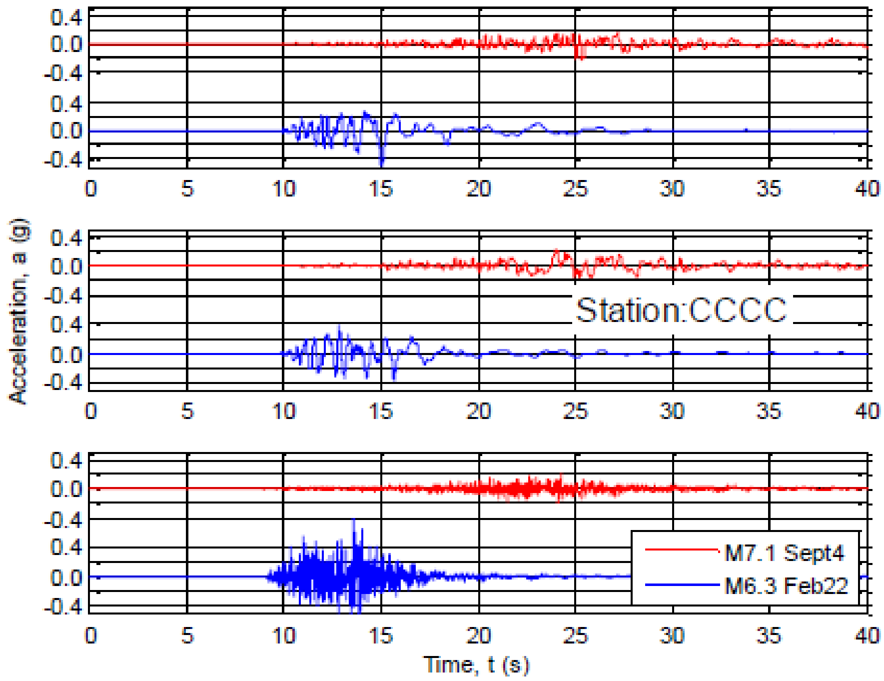

Figure 2. Recorded time histories from the Christchurch Cathedral College (CCC) site

2 Summary of structural observations

Although the strong ground motion only lasted about 10 seconds, the intensity of the vibrations was such that many structures were severely damaged. An overview of the impact of the earthquake on the built environment is given in Figure 3 (Dept. of Housing and Building, 2011). Failures of URM commercial buildings were common. Many iconic heritage structures were severely damaged including the much loved Christchurch cathedral. A significant number of reinforced concrete buildings suffered moderate or severe damage. From the 1980s onwards many reinforced concrete buildings were designed using capacity design principles, a design approach that leads to controlled structural yielding (in a ductile manner) during a major earthquake (and hence damage). The catastrophic collapse of two non-ductile reinforced concrete buildings, the CTV and Pyne Gould, had a high consequence in terms of fatalities. In contrast, some low-rise reinforced concrete buildings in the Christchurch CBD suffered very little damage. However, the presence of heavy masonry elements that were poorly attached to the structure, or of plan irregularities that induced a strong torsional response, resulted in considerable damage in some of these low-rise buildings. Timber and steel buildings typically performed well, as did some reinforced masonry ones, although not those of the older type. Reinforced concrete rather than steel was the dominant building material for commercial buildings during the 1980s property boom in Christchurch. Hence most of the steel buildings that are in existence, including concentrically and eccentrically braced frames and moment-resisting frames, have typically been designed to modern codes. An excellent summary of the effect of the earthquake on steel structures in Christchurch is given in (Bruneau et al, 2011).

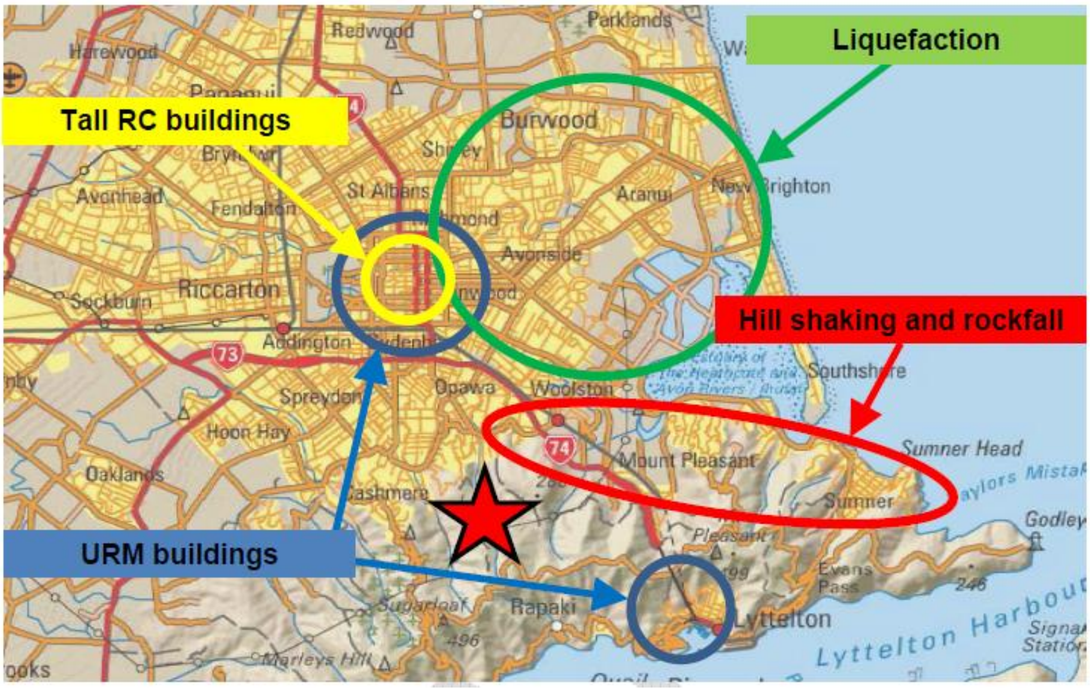

Figure 3. Overview of the impact of the earthquake on the built environment

In general, there was a significant correlation between the age of the structure and the extent of damage. No loss of life occurred in any building constructed after 1991. One reason given for this correlation is that "whenever the seismic loadings requirement has been increased, it has not been a retrospective requirement in New Zealand to strengthen existing buildings" (IPENZ, 2011). The most recent Building Act in New Zealand, which came into effect in 2004, required territorial authorities to develop an earthquake-prone buildings register, compulsorily including buildings for which the assessed strength was less than 34% of the Building Code’s requirements for a new building (IPENZ, 2011). Each territorial authority was allowed to set the time scale within which the building owners had to act to reach this minimum level. At the time that this Act came into being, the New Zealand Society for Earthquake Engineering indicated that this was not stringent enough; they (supported by IPENZ) strongly recommended that the minimum requirement for any strengthening undertaken be set at double this requirement, i.e. 67% rather than 34%.

Liquefaction and lateral spreading were the predominant geotechnical characteristics of the earthquake; the Christchurch earthquake not only re-liquefied areas that had been affected in the Darfield earthquake, but also caused a more widespread liquefaction. Lateral spreading that occurred close to the local rivers caused damage to some of the bridges.

3 Response Spectra

The 5% damped elastic spectral acceleration and displacement response spectra (Kam et al, 2011) for 4 different sites in the CBD are shown in Figure 4 and Figure 5 respectively. Also shown is the Ultimate Limit State earthquake design response spectra for soft soil (Class D) and for buildings of normal importance (representing the 500 year return period) from the current New Zealand earthquake loading AS1170.5; this is based on an assumed PGA of 0.22g for Christchurch. The comparable design response spectra from the 1976 NZ loading code is shown to be much less demanding. The current loading code's 2500 year return period event is also represented; it is derived simply by multiplying the 500 year return period spectra by 1.8. The E-W direction is the predominant one, although the N-S direction is still significant (usually only 15 to 30% lower) and it even exceeds the E-W spectra in the 0.35 to 0.6 period range. In the E-W direction the acceleration spectra matched or exceeded the typical NZ 2500 year return period earthquake in the 0.5 to 1.75 second period range.

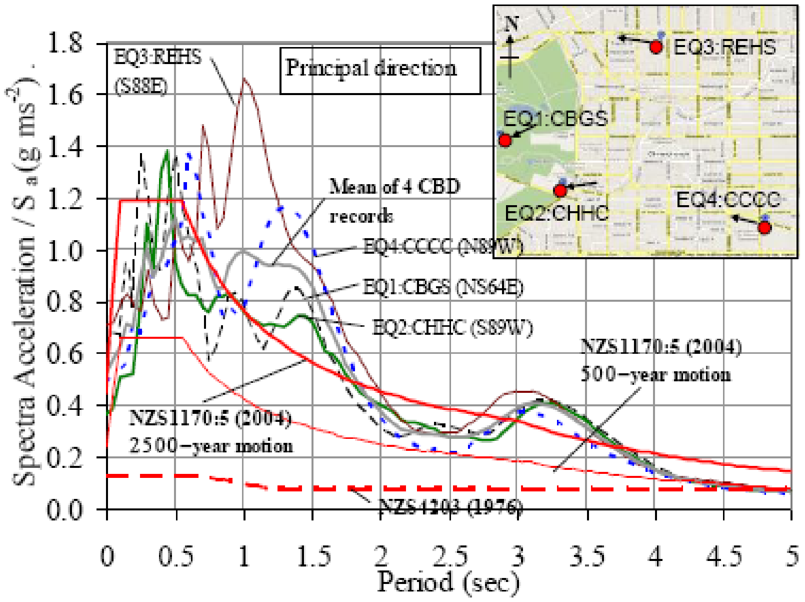

Figure 4. 5% damped elastic spectral acceleration response spectra for 4 sites in the CBD

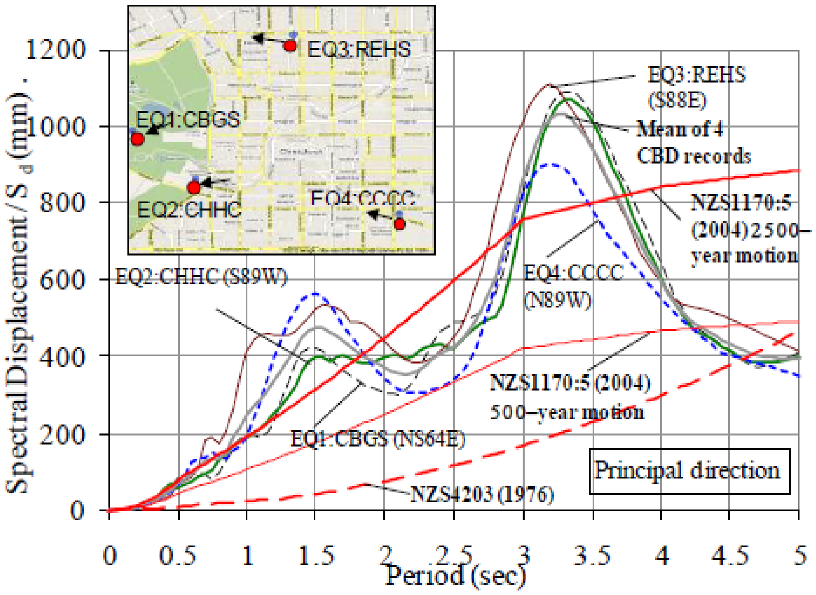

Figure 5. 5% damped elastic spectral displacement response spectra for 4 sites in the CBD

Throughout the period range relevant to building structures, shaking in the CBD exceeded the 500 year typical ULS design response spectra. Hence, an increase in the required retrofitting strength from 34% to 67% of the Building Code's requirement for a new building may still have been insufficient if it relied on strength alone. (SeSoc, 2011) states that "Buildings strengthened to the minimum standard (new systems, up to 33%) achieved life safety performance, but will still be demolished in many cases. Buildings strengthened to a higher standard (new systems, 67% or more) performed better - many survived with moderate to low damage. From this, we would conclude that 33% of code is still too low in most cases."

There was, in general, considerable spatial variation in the spectral values. For example, the spectra obtained using records from the four recording station within the CBD itself can be seen to be quite different in Figure 4 and Figure 5. According to (Smyrou et al, 2011) the highest spectral values for short period motion tended to be close to the epicentre of the earthquake whereas those for higher period motion were close to the CBD. They think that this can be partly attributed to potential source effects, but may also be due to soil softening, in particular due to soil liquefaction. They state that "response spectra obtained in liquefied areas are often characterised by bulges in the long-period range". Bulges of this type were observed in spectra from both the Darfield and Christchurch earthquakes; the bulge in the acceleration spectra occurred at 2.5 sec to 3 seconds for the Darfield earthquake and at approximately 1 to 1.5 seconds and at 3 seconds for the Christchurch earthquake. Other researchers (Galloway et al, 2011) have simply referred to the "significant amplification of long period response due to the depth of underlying gravels" and have questioned whether "the response of the Canterbury Plains gravels is well modelled by the current spectral shape for Class D (deep) soils specified in NZ1170.5:2004".

4 Non-ductile reinforced concrete buildings

The pre-1970s reinforced concrete buildings in the CBD are typically non-ductile. The response spectra indicate that mid-rise to tall structures in the CBD are in a period range that made them particularly susceptible in this earthquake. (Smyrou et al, 2011) describe a "typical mid-rise RC concrete building" in the city as one that has been built with RC walls around stairs and lift shafts. They say that studies on such structures, where RC walls have not been built with the primary concern of earthquake resistance, indicate that the following simple formula gives a reasonable approximation to the yield period of these structures:

$$T_y = 0.075\ H \tag{1}$$

where H is the total height in metres.

A similar formula is given in (Priestley et al, 2007) for both walls and frames, using 0.1 instead of 0.075. Code estimates of the period are usually lower, and this is justified on the basis that it allows for the contribution to the stiffness made by "secondary elements" such as gravity frames, or precast facades or masonry infill. It is usually considered conservative to use a lower period when determining the design acceleration from the response spectra. However, as can be seen in Figure 4, in the case of the Christchurch earthquake, structures in the CBD could have been subjected to even higher accelerations if they softened when "secondary elements" failed.

Using equation (1) for a 5 storey building with 3 metres floor-to-floor heights, T_y = 1.1 seconds. It can be seen in Figure 4 and Figure 5 that this is in a range of high spectral values for both accelerations and displacements. At the REHS site, if the corresponding substitute structure behaved elastically it would have been subjected to a 450 mm displacement at the centre of mass (and an acceleration of 1.5g), hence approximately 675 mm at the top of the building. If evenly divided between the five levels this would cause a 4.5% drift. This simple calculation indicates that the building would have yielded (yield usually occurs at less than 1% drift for a building of this type). It should be noted that yielding would have led to a higher level of effective damping than 5% and the displacement and acceleration of the substitute structure would have been less than estimated above. Nevertheless these simple calculations are indicative of the extreme deformations and forces such buildings would have been subjected to.

In the case of the five-storey Pyne-Gould building which was built in 1963 to 1964, the sequence in which the failure occurred has been investigated thoroughly and reported in a preliminary report from the Royal Commission. It is shown in Figure 6 (Beca, 2011). In that case the longitudinal reinforcement in the western core wall fractured in tension followed by a catastrophic compressive failure of the eastern wall. The columns, and/or joints between the columns and the beams, in the non-ductile perimeter frames (with no ties in the joints) then failed due to excessive lateral drifts. The slab-to-wall connections were unable to withstand the rotations, shears and tensions resulting from the forced displacement of the shear core, leading to a sequential collapse of the floors. Photos of the collapsed Pyne Gould building are included in Figure 7 and Figure 8 (Kam, 2011b).

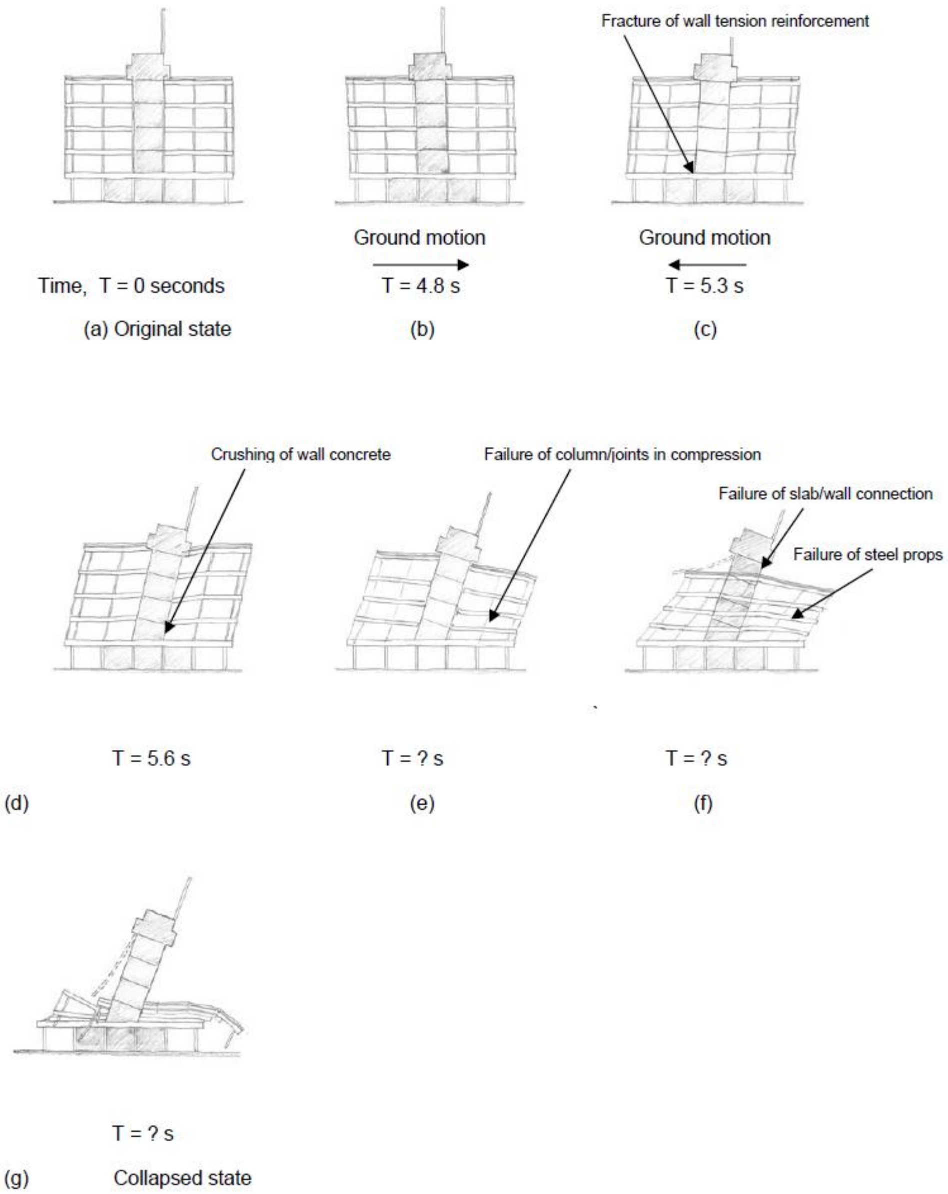

Figure 6. The failure sequence of the Pyne-Gould building

Figure 7. Pyne Gould collapse from the southern side

Figure 8. (a) Detail of joint failure in the Pyne Gould perimeter frame, (b) Collapsed Pyne Gould building from the northern side

5 Modern concrete buildings

As discussed previously many of the modern concrete buildings sustained some damage, something that was not unexpected given the prevailing design philosophy, that of "capacity design". Many observations have been recorded and discussed in various reports by New Zealand engineers, some of them written for the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes (including (SeSoc, 2011), (Buchanan et al, 2011), Dept. of Building and Housing (2011))

Some important conclusions are listed here:

- Although capacity designed buildings usually behaved as expected by structural engineers, there is a disconnect between societal expectations and the actual extent of damage and disruption. Concerns are also expressed about low cycle fatigue and the current ability of engineers to assess the reliability of buildings that have experienced several cycles of plastic deformation in the plastic hinge regions. More research is needed (SeSoc, 2011) in this area. Something that has compounded this problem is the field observation that yielding in the plastic hinge regions was often concentrated in one or two wide cracks rather than being distributed within several narrow cracks as observed in laboratory tests. In some elements this was observed to lead to fracture of the longitudinal bars as shown in Figure 9 (Buchanan et al, 2011). This is probably due to concrete strengths being much higher than the design characteristic values, or possibly to the R_u to R_e ratio (the strain hardening ratio) of the reinforcement being unacceptably low. The estimation of the extent to which the cyclic plastic capacity of the bars has been exhausted is critical in determining whether low-cycle fatigue could be an issue in future earthquakes.

Figure 9. (a) Small cracks in base of a tall wall. Note the minor damage at far end of wall. (b) Damaged end of wall after breaking out some concrete. The vertical bars have yielded then fractured.

- Compressive failures in walls were often excessive with loss of concrete and buckling of reinforcing bars in the plastic hinge zones as shown in Figure 10 (Buchanan et al, 2011). These observations indicate that additional sources of loading need to be considered and the confining steel may need to be spread out over the length of the wall rather than just concentrated at the end zones. Importantly, according to (SeSoc, 2011) "It is debatable whether the current practice of ignoring out-of-plane movement of concrete walls may be unconservative. There may be cases where the combination of in-plane and out-of-plane movements has caused failure of walls, including possibly the Grand Chancellor shear wall." (see further discussion below)

Figure 10. Compressive failures in walls. Loss of concrete and buckling of reinforcing bars in the plastic hinge zones

- Vertical accelerations were thought to be a factor in the failure of the shear wall in the Hotel Grand Chancellor building (see further discussion below). (SeSoc, 2011) cautions against overemphasising this factor since the high vertical accelerations were typically very short period transient effects. Nevertheless they are also concerned about transfer structures where an increased vertical load may result in a disproportionate impact.

- Floor elongations due to the formation of plastic hinges in moment-resisting frames has been observed to lead to substantial tearing of floors, sometimes across the full width of the diaphragm, often fracturing the brittle mesh within the slabs. This can potentially result in the loss of a force transfer from the floor to the lateral force resisting system. Precast slab systems are especially vulnerable since they often have pre-existing floor cracks due to shrinkage and creep; also the frame elongation can reduce the length of seating available for the precast elements (Fortunately, in this earthquake, no precast floor slabs actually collapsed due to loss of seating (Buchanan et al, 2011).

- Given that stairs are sometimes the only means of emergency egress, stair failures have been subject to close scrutiny. In the conclusions of the Expert Panel Report (Dept. of Building and Housing, 2011) relating to the collapse of the stairs in the Forsyth Barr building, the Panel states "The seismic gap specified on the drawings met the design standards prevailing at the time the building was designed." They go on to say that "The specified gap would not have been sufficient to avoid compression if the current (2010) code-derived displacements had been applied". The gaps are very necessary and yet it was also concluded that "it is possible that the seismic gaps at the lower supports had been filled with material that restricted movement (including debris, mortar or polystyrene) which reduced their effectiveness."

The results of the investigation into the collapse of a shear wall in the Hotel Grand Chancellor Building by the Royal Commission's Expert Panel (Dept. of Building and Housing, 2011) exemplify some of the points made above. This 22-storey hotel was built between 1985 and 1988. The collapsed 4.5 metre long RC shear wall is shown in Figure 11 (Kam, 2011a). Their findings are summarised as follows:

"Extremely high compression loads combined with low levels of confinement reinforcing led to the wall failure. The lapping of vertical reinforcing and the slenderness of the wall also appear to have contributed to the onset of failure. Under the action of high compression loads, a small transverse displacement was enough to initiate failure in the unconfined concrete. The high axial loads arose from the building geometry and induced actions resulting from the severe horizontal accelerations. It is highly likely that vertical accelerations contributed to the high compression loads."

Figure 11. Collapsed Shear Wall in Hotel Grand Chancellor

6 The effects of liquefaction

As mentioned previously, Christchurch is situated on deep alluvial deposits and sites are generally classified as level D (deep or soft soil) for structural purposes in the New Zealand Design Standards (NZS 1170.5:2004). According to (Tonkin and Taylor, 2011) there is a geological formation that underlies many areas in Christchurch called the Springston. Formation and alluvial deposits within that formation include deposits of silt and sand that are highly susceptible to liquefaction. Although the Christchurch earthquake was of short duration, the ground accelerations were sufficiently high to cause much of the saturated loose sands and silts below the water table to liquefy leading to some spectacular effects including major ground settlement, lateral spreading and foundation support failure. The effects on bridges and buildings have been documented by several researchers (Chouw et al (2011), Smyrou et al (2011)). Some photos taken by the author are included to highlight some of the major problems; lateral spreading (Figure 12), flotation of pipes (Figure 13), rotation of bridge abutments around the bridge deck leading to localised crushing of the bridge deck and buckling of the roadway (Figure 14), and subsidence of fill behind the bridge abutments (Figure 15). Figure 16 is a photo (Kam, 2011a) which illustrates the tilting effect of liquefaction on a five storey concrete frame building on shallow foundations close to the Avon river.


Figure 12. Effect of lateral spreading on the roadway


Figure 13. Flotation of pipe below causes the manhole to rise


Figure 14. Rotation of bridge abutments and buckling of roadway

Figure 15. Subsidence of fill behind the abutment

Figure 16. Tilting of building caused by liquefaction (Photo by Wen Kam)

7 Effects of vertical accelerations

Pounding due to high vertical accelerations was an issue in bridges. This caused crushing of the cover concrete in bridge piers in one case. In another, the edge of a long wall support was knocked off at the top due to intense localized vertical forces from the bridge bearings (as the wall pounded on the bridge deck from below). This is shown in Figure 17.

The effects of vertical accelerations on buildings have been discussed above.

Figure 17. Crushing of the cover concrete in a bridge pier

8 Lessons from the Christchurch earthquake

Although there was considerable damage in the Darfield earthquake of September 2010, there were no fatalities. The Christchurch earthquake had a much greater impact on the psyche of the people. Just when they had thought the worst was over, they were subjected to a significantly more damaging event, and one with 181 fatalities. It was the horror of people losing their lives that shocked the general populace more than anything else.

Christchurch is in a zone of moderate seismicity. The codified design level (500 year return period) PGA for Christchurch is half that of Wellington and 1.5 times that of Auckland. Nevertheless, as mentioned previously, the ground motions recorded in the Christchurch earthquake were very high in the Christchurch CBD, causing response spectral accelerations and displacements in excess of those codified for a 2500 year return period earthquake over a significant portion of the building-relevant frequency range. (SeSoc, 2011) have commented that "In many cases the effect of deep alluvial soils appears to have been underestimated. Although the subsoils clearly complied with the description of Class D in AS/NZS 1170.5, there has been significant amplification of the high period response in some cases. It is not known if this is unique to Christchurch, but [it] needs further research."

Capacity design principles and detailing methodologies, as applied to reinforced concrete in particular, were developed at the University of Canterbury by Robert Park, Tom Paulay and their successors, Nigel Priestley, John Mander and others. It is largely because of those developments and the implementation of them by practicing structural engineers in New Zealand, that the number of fatalities in the Christchurch earthquake was not greater. The plastic deformations of hinge regions in many of the medium-rise to high-rise reinforced concrete buildings enabled the buildings to deform without collapse in this extreme event. It appears, however, that the general public has been underwhelmed by the end result: the extent of the damage, the cordoning off of the CBD, the societal and economic effects of the disruption to services and business activities, and most importantly, the deaths of 181 people, were worse outcomes than the general populace had anticipated.

The poor performance of some of the older buildings was expected and recommendations have been made to increase the required strengthening from 33% to 67% of the code requirement for new structures. Some engineers are also dissatisfied with the philosophy of capacity design and the extent of the damage suffered by some modern buildings in the Christchurch earthquake (Buchanan et al, 2011). They would like to implement new design technologies to improve the seismic performance of buildings even under extreme events. Their proposed Performance Objective Matrix is given in Figure 18 (Buchanan et al, 2011) and is intended to replace the current performance matrix given in Figure 19 (Buchanan et al, 2011). They propose a combination of two different tactics: that of increasing the level of seismic design loading, and that of switching to higher performance building technologies such as base isolation and damage-resistant technologies. These damage resistant design technologies are on the cusp of earthquake resistant design inventiveness and include rocking walls and frames, with and without post-tensioning, and a variety of different energy dissipating devices attached to the building in different ways. According to (Buchanan et al, 2011) "If not already the case, damage-resistant design will soon become no more expensive than conventional design for new buildings." These technologies have already been implemented in New Zealand at the reinforced concrete Endoscopy building at Southern Cross Hospital in Christchurch, TePuni Village steel building at Victoria University in Wellington and the new NMIT timber building in Nelson.

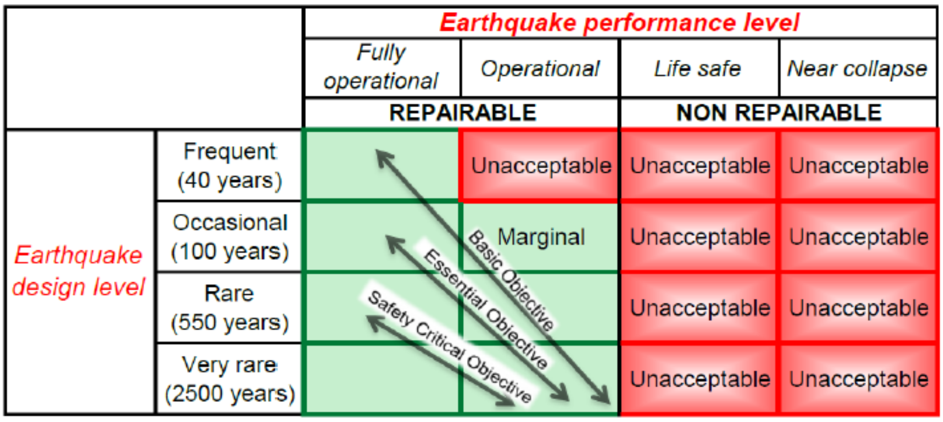

Figure 18. The proposed Performance Objective Matrix

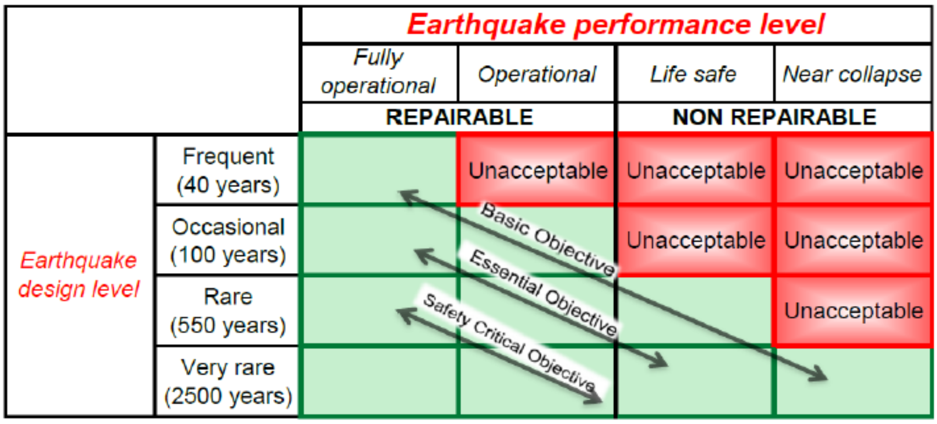

Figure 19. The current Performance Objective Matrix

(SeSoc, 2011) notes that base-isolation has often been used successfully in overseas practice and tries to give possible explanations as to why it has not gained greater popularity in New Zealand. They also observe that "the impact of deep soft soils [in Christchurch in particular] also needs careful consideration with the potential increased accelerations in the long period range." With regard to what they call "low damage design" they are concerned that "we do not inadvertently swap the mistakes of the past for new mistakes in our eagerness to move forward." They caution against rushing into the widespread adoption of these new "low damage" technologies without recognising their short-comings. They say that "Concern should be expressed about PRESSS systems in any material which do not address beam elongation issues that are potentially just as severe as in ductile moment frames."

In the Christchurch earthquake even if someone owned a building in the CBD that suffered only minor damage, this building would have been out of operation for many months after the earthquake. This was because of the extreme damage suffered by many buildings and doubts about their safety which led to the CBD being cordoned off. If damage-resistant design is achievable and is widely employed quite a different post-earthquake scenario would be expected. In particular, the minimum performance objectives for an ordinary building at importance level 2 would be very similar to that of the importance level 3 and 4 buildings, even for very rare earthquakes. Hence, in all cases the buildings would be expected to be repairable, and widespread closures of entire city areas would be rendered unnecessary.

In Australia, the current design level earthquake for our capital cities is low. The Building Code of Australia stipulates the return period for design level earthquake design to be 500 years for ordinary buildings (importance level 2), with a higher return period for "more important" buildings, eg. 1500 years for buildings of importance 4. The design is typically "force-based" and the ULS (500 year return period) PGA in the capital cities is a bit less than half of that in Auckland. The code-compatible ULS displacement design spectrum for a soft soil site is said to have a T_2 value of just 1.5 seconds and hence a maximum value of 150 mm (much less than the 1.2 metres realised at some sites in the Christchurch CBD). One key point is that, unlike in New Zealand, designers are not forced to consider how the building would respond under a very rare event unless the building is deemed to be a very important one. Due to the lack of thought given to strength hierarchies within a building and the failure to incorporate weak ductile zones that allow the building to safely deform into the plastic range, the performance of some buildings is likely to be poor. The robustness clause in the loading code AS1170.0 is intended to ensure that the damage caused by an event is not disproportionate to the magnitude of that event. The question here is what type of damage would the structural engineering community view as appropriate for a very rare earthquake event. It is the author's opinion that "prevention of collapse" is the minimum performance that should be considered.

More research is needed by Australian seismologists to determine the ground motions in the various capital cities that are likely to be experienced in a very rare event, especially in zones with deep soft soils. Australia, although in a region of low to moderate seismicity, is one of the most active intraplate regions in the world. Most of the Australian capital cities have faults in their vicinity that are capable of generating damaging shallow earthquakes, and it is expected that an magnitude 6 or higher earthquake is likely to occur every few thousand years close to an Australian capital city. Is this the level of "very rare" earthquake that should be considered, i.e. perhaps one with a 10,000 year return period. Due to the vulnerable nature of the building stock, if such an earthquake did occur, it would be likely to cause extensive damage and a large number of fatalities. This is recognised by the insurance industry which has assessed that Sydney presents one of the highest earthquake risks in the world. It is unlikely that the Australian public is fully aware of this risk.

9 Conclusions

A general summary of the damage caused by the February 22nd Christchurch earthquake has been given with considerable emphasis on the behaviour of reinforced concrete buildings, and some emphasis on the effects of liquefaction and vertical accelerations. Some types of damage have not been discussed here: damage due to rock falls, damage to masonry buildings (both unreinforced and reinforced), pounding of one building against a neighbouring building or of a bridge deck against its abutment, failure of glass windows and others. Many of these have been covered in the detailed report produced by (Chouw, Hao and Goldsworthy, 2011) or by other authors.

Any major earthquake event leads to a reassessment of design philosophies, and the Christchurch earthquake is no exception to that. The most important lesson for Australia is that structural engineers need to be educated to incorporate reliable ductility in buildings so as to enhance the building's ability to withstand a very rare earthquake event without collapse. Displacement-based methods to assess building performance when subjected to a very rare earthquake event should become a routine part of the structural design. This approach will lead to more resilient types of building construction (with better details) being favoured by designers, and the adoption of newly developed forms of construction including "low damage" solutions for some buildings of high importance. Consideration of these issues will result in a building stock which will be more robust than it is now when and if a major earthquake does strike one of the Australian capital cities in a hundred years time or more.

It is anticipated that if engineers were forced to make these considerations, cost increases would be likely to be marginal. In (Sesoc, 2011) it is said that the cost of multi-storey office buildings are marginally more expensive in Auckland than in Wellington or Christchurch, but are all within 2% over a range of building types. They think that this suggests that regional material and labour cost factors more than compensate for any cost difference as a consequence of seismic loading.

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