# 4 THE NEED FOR DAMAGE-RESISTANT DESIGN

This chapter details the need for damage resistant design of new buildings, by giving a summary of serious damage observed in modern buildings in the 2010 and 2011 Christchurch earthquakes.

As well reported elsewhere, the February event had an extremely high level of shaking, significantly more than the design level earthquake, with vertical accelerations being among the highest ever recorded. This high level of shaking led to very high inelastic behaviour and severe displacement and deformation demands on a large number of buildings, and many of these will have to be demolished because of the excessive cost of repair. Many others have suffered significant business interruption and downtime costs. Given the high levels of recorded accelerations, the damage to buildings caused by ground shaking in Christchurch was largely as expected by structural engineers, because modern design standards encourage design for ductility, leading to controlled damage but avoidance of collapse.

Most of the modern buildings in central Christchurch are reinforced concrete, and a summary of damage to these buildings is given by Pampanin et al. (2011), highlighting a large amount of localised damage, especially in plastic hinge regions, exposing the limitations of traditional design philosophies not yet embracing a damage-control objective. A description of critical structural damage to non-residential buildings, and recommended assessment procedures, is given in a draft report by the Engineering Advisory Group (EAG, 2011).

The most important observed damage to structural components (excluding non-structural damage) includes:

- Major damage to plastic hinge zones of structural concrete walls including:
    - Loss of concrete and buckling of reinforcing bars.
    - Fractured reinforcing bars despite very little cracking of the surrounding concrete.
- Large inelastic deformations in moment-resisting frames, with plastic hinges and frame elongation, causing:
    - Serious cracking in concrete floor diaphragms, with fractured reinforcing bars.
    - Loss of seating of precast prestressed concrete floors.
- Fracture of welded steel in eccentrically braced structural steel frames.
- Excessive lateral displacements to parts of buildings, leading to:
    - Structural damage to frames which are not part of the lateral load resisting system.
    - Loss of support to stairs and ramps.

Most of this damage has required urgent repair, or demolition if the repairs are uneconomical. Even if the buildings are able to be re-used, there is often doubt about the residual ability to resist further major earthquakes. The solutions suggested below are for new buildings. Repair and reinstatement is not covered in this report, except in passing.

## 4.1 Serious Damage to Concrete Walls

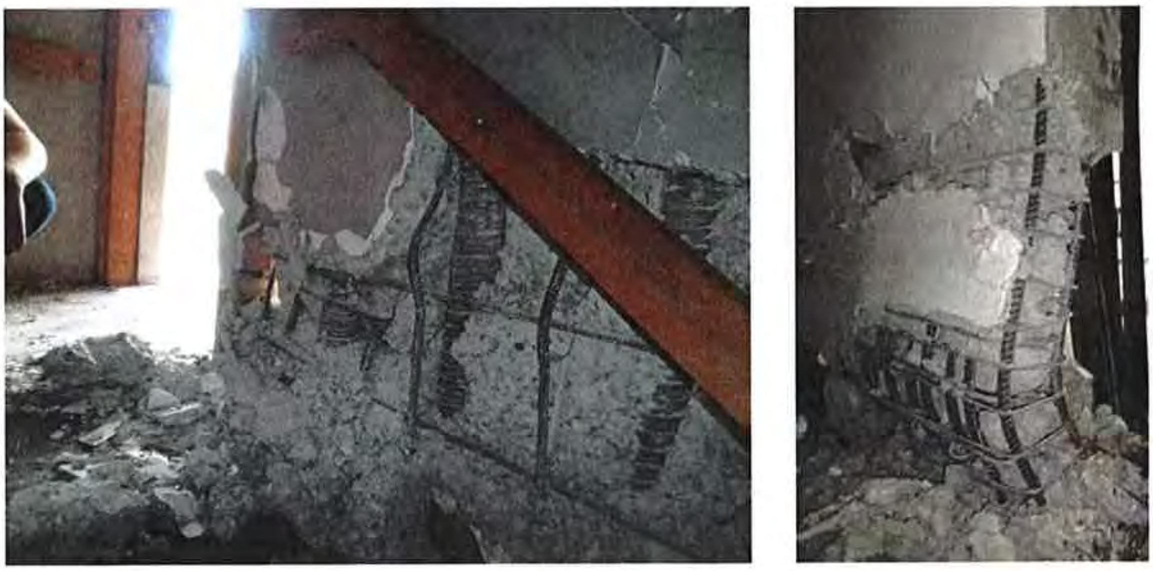
### 4.1.1 Loss of concrete and buckling of reinforcing bars

Some buildings have suffered severe localised damage to structural walls that are holding the whole building up. This severe damage has often been in the lower stories where flexural and shear stresses are highest, as shown in Figure 4.1 and 4.2. Traditionally, the strength and performance of concrete in compression has been improved by confining the concrete in the critical regions with closely spaced hoops or stirrups of reinforcing bars. The observed damage shows that insufficient confinement was provided in many cases.

Thin walls have performed much worse than expected, largely due to insufficient confinement reinforcing bars. This may be partly because of oversight in the design, or just the practical difficulty of fitting high concentrations of reinforcement into areas that will be highly stressed.

**(a) Photo from street.** **(b) Severely damaged end of structural wall.**

**Figure 4.1 Seven storey reinforced concrete office block.**



**(a) The system for joining precast concrete wall panels fails through insufficient confining reinforcement.**

**(b) Severe damage to reinforced concrete wall, with local buckling at the toe of the wall.**

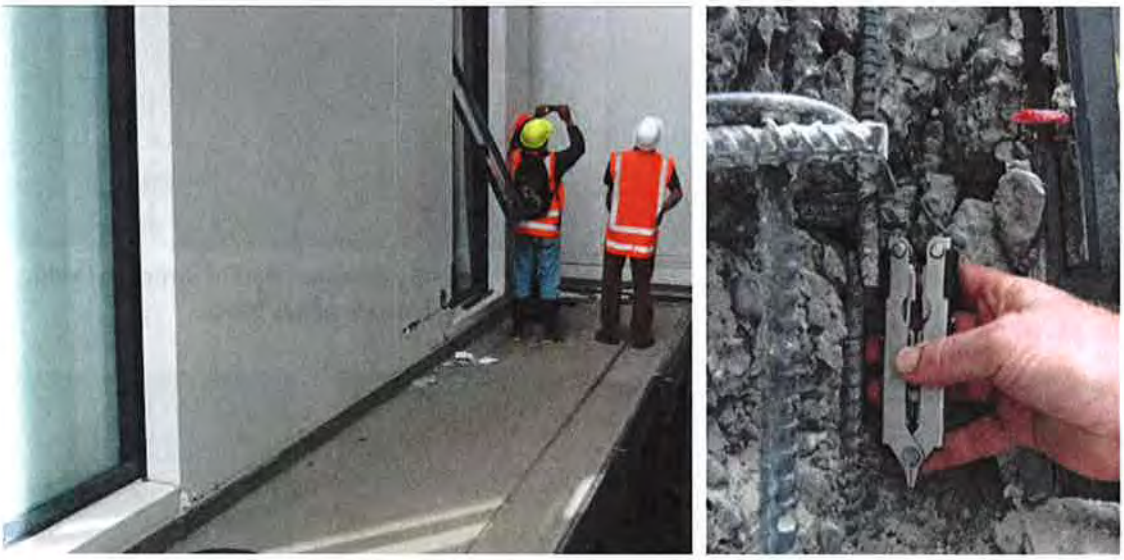
**Figure 4.2 Damage to reinforced concrete walls .**

Solutions:

- Use damage-resistant design. For example, design walls which will rock back and forth on the foundations under extreme lateral loading.
- Design buildings to avoid flexural plastic hinges in structural concrete walls.
- Provide more confinement in critical regions of structural concrete walls.
- Do not allow very thin structural concrete walls to be used.

### 4.1.2 Fracture of reinforcing bars in walls

Some semi-destructive investigation of structural walls in tall reinforced concrete buildings has identified a major problem of fractured reinforcing bars inside concrete elements that only show small cracks. An example is shown in Figure 4.3. This type of damage is due to the relatively small amounts of reinforcement in the walls, and a much higher concrete strength of the aged element than specified by the original designers. Extensive laboratory testing of reinforced concrete structures in New Zealand and around the world has shown that "plastic hinges" in beams and columns usually have a widespread pattern of cracks in the concrete, so that stresses in the internal reinforcing bars are distributed over a significant length of adjacent concrete.



**(a) Small crack 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.**

**Figure 4.3. Yielding and fracturing of wall reinforcing steel in a tall building.**

If the concrete in a real building is much stronger than that tested in the laboratory, only one crack (rather than an array of cracks) occurs in the critical region, placing excessive strain demands on the reinforcing steel and sometimes leading to fracture of the bars. Many buildings have critical cracks which had clearly opened several centimetres during the earthquake, enough to fracture the bars, before closing up due to gravity loading after the shaking subsides. These fractured bars are hard to find, so there may be many more in damaged buildings that are undetected. Solutions for new buildings are not straightforward; simply placing more steel bars in the walls is not a solution because it will increase the strength of wall, in precisely the location where the intended "weak link" is supposed to be (this is a region of wall that is meant to "yield", or deform plastically; the "plastic hinge").

Solutions:

- Use damage-resistant design. For example, design buildings to avoid flexural plastic hinges in structural walls.
- Place upper and lower bounds on the strength of concrete in plastic hinge regions.

## 4.2 Large Deformations in Moment-Resisting Frames

Large inelastic deformations in moment-resisting frames result from plastic hinges occurring in the beams. Plastic hinge deformations often result in considerable lengthening of the beams, called "frame elongation". The effect of frame elongation is to cause the building to bulge or balloon out as described later in Chapter 6. This frame elongation effect causes several problems as outlined below (Peng, 2009).

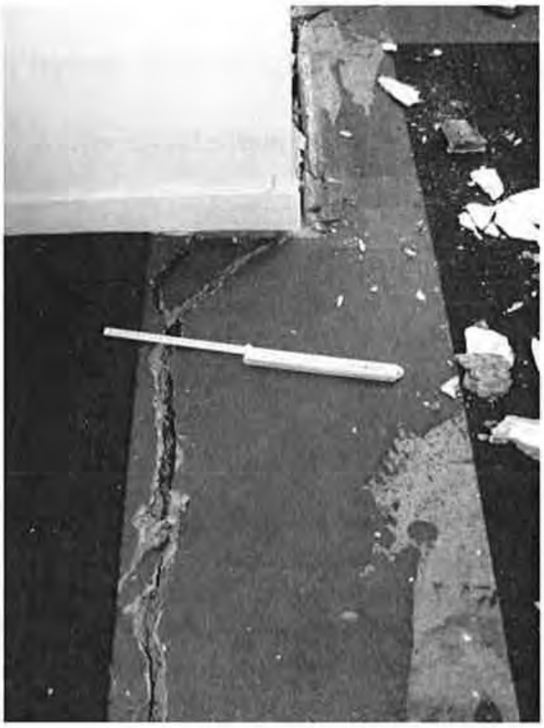
### 4.2.1 Damage to floor diaphragms

Many buildings have suffered severe damage to reinforced concrete floor diaphragms. The main reason for this damage is frame elongation, with columns being forced apart by the formation of "plastic hinges" in the beams of moment-resisting frames (Figures 4.4and 4.5). This results in the whole building growing a bit bigger during the earthquake, causing major cracks in floor slabs, or in the topping concrete on precast concrete floor slabs. Reinforcing bars are often fractured in the cracked region. The initial concern about this cracking is the loss of the floor diaphragm action which holds the whole building together and transfers seismic forces to the lateral load resisting system.

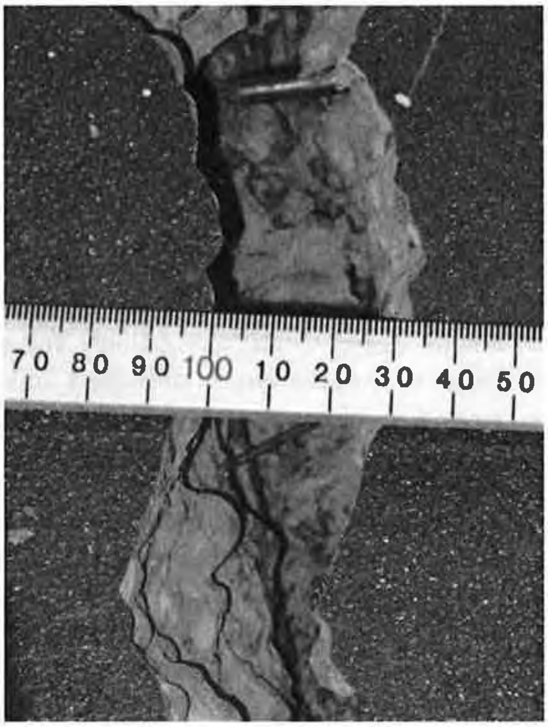
Solutions:

- Use damage-resistant design. Design buildings without ductile moment-resisting frames.
- Find other ways of achieving ductility and hence dissipating seismic energy.
- Require larger amounts of reinforcing in topping concrete.
- Avoid the use of non-ductile welded wire reinforcing mesh.

**Figure 4.4. Plastic hinges at ends of beams in reinforced concrete frames.**



**(a) Frame has elongated and moved away from the precast concrete floor**




**(b) Detail of the crack between the beam and the floor. Cold-drawn wire mesh has fractured.**

**Figure 4.5. Damage to floor slabs in a multi storey concrete building.**

### 4.2.2 Seating of precast flooring systems

Most modern buildings in New Zealand have precast prestressed concrete floors with cast in-situ reinforced concrete toppings (50–75mm thick). These floors are usually simply supported one-way spanning systems, although flexural continuity is sometimes provided by placing additional reinforcing bars in the topping over the internal supports (the beams and walls).

Traditionally, the length of seating at the ends of precast prestressed concrete floors has been insufficient. Therefore when the building grows and the slab is damaged due to frame elongation as described above, the seating becomes marginal, as shown in Figure 4.6. It is extremely fortunate that no floor slabs actually collapsed in the earthquakes, although some of the observed stair collapses may have been from this cause.

Solutions:

- Design buildings without using ductile moment-resisting frames.
- Find other ways of achieving ductility.
- Require much larger seating for precast floor systems.
- Consider using two-way cast-in-place reinforced concrete floors.

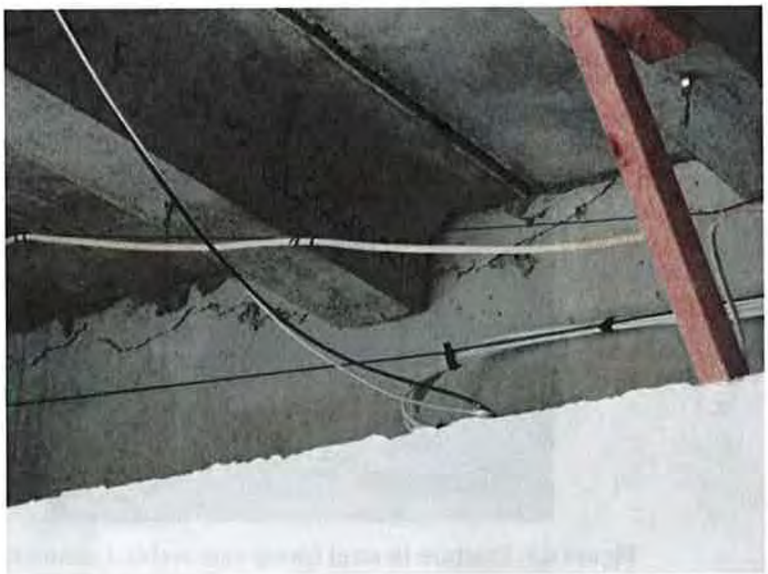


**Figure 4.6. Spalling of concrete ledge supporting the flange-hung Tee units.**

### 4.2.3 Low cycle fatigue in reinforcing bars and structural steel

"Low cycle fatigue" refers to the fracture of steel due to a small number of strain reversals, well beyond the elastic strength of the steel. The best analogy is "a paperclip bent at right angles and bent back straight, a number of times, until the paperclip breaks". This is what happens to reinforcing bars that are stretched and compressed well beyond their yield strength during a seismic attack. A small number of big strains will cause the bar to fracture. This is called "low cycle fatigue".

This fracturing is hastened when the bars are bent sideways as the beams, columns or walls are damaged. Typically, visual inspection is not able to determine if steel bars or other steel members are close to fracture. Testing of samples is needed.

Steel members and steel bars that may have used up most of their plastic deformation capacity (and may be near fracture, in some cases) will be very difficult to find and repair. Repair will not be feasible in a lot of circumstances. This is elaborated upon in the next section.

Solutions:

- Design all structural members and their connections to avoid accumulative plastic strains.
- Design and install easily replaceable components that undertake the accumulative plastic strains needed to absorb energy from the earthquake.

## 4.3 Fracture of Welded Steel Members

Structural steel is normally considered to be a very ductile material. However, one serious brittle fracture was reported of an eccentrically braced frame (EBF) in a hospital car-parking building, very close to an eccentric welded connection (Figure 4.7). Brittle fractures are not normally expected in steel structures, but it is known that welding of structural steel can cause strain-age-embrittlement leading to brittle failures. Some new research in this area may be needed.

Solutions:

- Provide guidance on welding procedures for localised regions intended to be ductile.

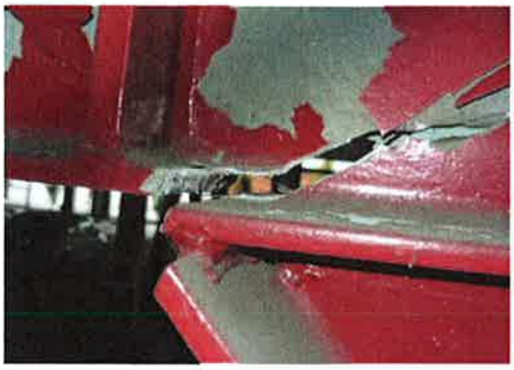


**Figure 4.7. Fracture in steel frame near welded connection.**

## 4.4 Excessive Lateral Displacement of Buildings

Many buildings had large amounts of lateral displacement caused by earthquake shaking. In some of these buildings, the lateral load resisting system performed well, but the displacements caused structural damage to frames and other structural components, or to stairs and ramps or other secondary structure.

### 4.4.1 Structural damage to frames which are not part of the lateral load resisting system

In some buildings with large amounts of lateral displacement, the lateral load resisting system performed well, but the displacements caused extensive and expensive structural damage to frames and other structural components which are not part of the lateral load resisting system.

### 4.4.2 Stair failures

Some failures of stairs and ramps occurred because of the loss of stair and ramp supports, through underestimation of lateral displacements of parts of buildings.

Solutions:

- Use damage-resistant design. For example, provide base isolation to limit the inter-storey movements during earthquakes.
- Ensure that the recommended limits for lateral displacement are met, both at the serviceability limit state and at the ultimate limit state.
- Stairs and ramps which span from floor to floor must have sliding joints which are designed to accommodate sufficient floor-to-floor movement.

## 4.5 Summary

Much of the damage described in this chapter could have been prevented or minimised by the use of new high-performance solutions for damage-resistant design, as described in the following chapters.

## References

EAG, (2011). Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-Residential Buildings in Canterbury. Part 2 - Evaluation Procedure (Revision 5). Engineering Advisory Group, Christchurch.

Pampanin, S., Kam, W.Y., Akguzel, U., Tasligedik, S., Quintana Gallo, P., (2011). " Seismic Performance of Reinforced Concrete Buildings in the Christchurch CBD after the 22 February 2011 Earthquake", Department of Civil and Natural Resources Engineering, University of Canterbury, report under preparation.

Peng, B., (2009). Seismic Performance Assessment of Precast Concrete Buildings with Precast Concrete Floor Systems. PhD Thesis, University of Canterbury.