INDEPENDENT ASSESSMENT ON EARTHQUAKE PERFORMANCE OF

151 WORCESTER STREET

FOR
Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes

Report prepared by Peter C Smith and Jonathan Devine
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Introduction

This report has been commissioned by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes to review the performance of the building at 151 Worcester Street, Christchurch, during the Canterbury earthquake sequence.

The report is based on documentation provided by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes and a limited inspection of the exterior of the building on the 9th September 2011 whilst the building was being demolished.

Location of Building

The building was located at 151 Worcester Street, Christchurch City, with Worcester Street to the south. To the west was a two-storey building at 145 Worcester Street, to the north was a car park, and to the east was the driveway access to the car park.

The location of the building in the Christchurch CBD is shown on an aerial photo of Christchurch included in Appendix 1, together with the direction of the epicentre of the main earthquakes.

Geotechnical Site Assessment

At the time of writing the report we have not had access to any geotechnical reports on the site of the building.

Description of Building

The building at 151 Worcester Street was a 7-storey building that had been constructed circa 1988. The building was square in plan and its structure consisted of two reinforced concrete frames in a north south direction and four reinforced concrete frames in the east-west direction. The ground floor level contained car parking and retail space, the remaining floors contained office space.

The building was designed by Alun Wilkie Associates Architects and DJ Falloon Consulting Engineer in 1987. We have reviewed the engineering drawing set (496/ S1 to S11) as provided.

The exterior cladding consists of precast concrete panels to the west wall, and timber-framed spandrel construction with windows to the other three sides.

Gravity System
The flooring was a Stahlton flooring system with a 75mm insitu concrete topping slab orientated so that spans ran north-south between the four bays of reinforced concrete frames. The topping slab was reinforced with D10 reinforcing.

A feature of the gravity frame to the north wall was that the three bay frame above first floor level was replaced with a two bay frame on the ground floor, with the north-eastern column located 1.6m to the south, back from the corner of the building.

Foundations

The concrete ground beams were supported on 500mm dia concrete bulb piles founded 10m below the ground floor level. The majority of external columns were supported on a single pile while the central column to the north wall and the internal columns were supported by 2 piles. The internal columns to the west wall were also supported on 2 piles.

Seismic System

The building was constructed with four concrete frames in the east-west direction, and two external frames in the north-south direction, with a precast concrete panel cladding wall on the west elevation of the building.

The lateral load resisting system to the building utilised the two internal frames in the east-west direction. The two external frames were gravity only frames. The columns to the internal frames were 800mm x 300mm reinforced concrete columns, with the columns to the north and south gravity frames being 350mm diameter, the majority being precast reinforced concrete.

The external frames to the eastern and western elevation were seismic resisting reinforced concrete frames.

The ground floor masonry walls were not relied on for seismic resistance.

The west wall of the building was located 130mm from the west boundary.

Floor diaphragm

Diaphragm action was provided by the Stahlton flooring system with a 75mm concrete topping slab.

Stairs

The main stairs were constructed from precast reinforced concrete and were supported using a structural steel frame and the main concrete frame of the building. Both the stairwell and the lift were located against the west wall of the building.

The details provided minimal provision for inter-storey deformations.

Compliance

The structural design calculations were sent to the Christchurch City Council on the 10th September 1987. The Christchurch City Council issued a building permit for the project on the 4th of November 1987. As the building was constructed prior to the introduction of the Building Act 1991, no code compliance certificate was involved.

In November 2008 a Certificate of Acceptance was issued by the Christchurch City Council for the realignment of a ground floor non-load bearing concrete block wall. In the Christchurch City Council Certificate of Acceptance worksheet associated with this application it is noted that the building was earthquake prone.

The building is assessed as complying with the requirements of the Building Act 1991 due to the building pre existing the Building Act and any alterations or change of use occurring since the introduction of the Building Act 1991, being approved by the Christchurch City Council.

Events Subsequent to 4th September 2010 Earthquake

No structural damage of note was recorded post the 4th September, 2010 earthquake or 26th December, 2010 earthquake. The building was severely damaged in the 22nd February, 2011 earthquake.

A Rapid Assessment Level 1 dated 5th September, 2010 assigned the building a green placard with the overall building damage noted as none.

Evans Douglas Consulting Engineers inspected the building on the 6th September, 2010, and they noted that there was cracking damage to gib-board of non-structural internal partitions, and some spalling to the underside of the stairs at the floor junction.

The building was again inspected by Evans Douglas on the 9th September, 2010 following a 5.1 magnitude aftershock. The inspection notes record that the building had suffered no structural damage, but that there may have been some further non-structural damage.

On the 20th October, 2011, Evans Douglas Consulting Engineers carried out another inspection following the aftershocks of the 19th October, 2010 and noted that they could see no fresh signs of movement or cracking. This letter goes on to note damage in the form of cracking to gib board sheet finishes, similar to that which had previously been recorded.

We have sighted no record of any damage that was sustained to the building following the 26th December, 2010 earthquake.

After the 22nd February, 2011 earthquake, the building was assigned a red placard on the 27th February, 2011 by Civil Defence Emergency.

The risk assessment on the 19th March, 2011 by Civil Defence Emergency noted a severe risk of the building collapsing, and an assessment of 75 to 100% overall building damage. The recommendation to address the damage was demolition of the building.

Beca Carter Hollings & Ferner Ltd carried out visual inspections of the building on the 28th March, 2011, 26th May, 2011, and the 2nd June, 2011. A report was produced by Beca Carter Hollings & Ferner Ltd "Earthquake Damage Assessment – 151 Worcester St", dated 27th May, 2011.

A further risk assessment was undertaken on the 14th June, 2011 by Civil Defence Emergency, after the 13th June, 2011 earthquake, and yielded a similar report to the 19th March, 2011 report.

Structural Performance

Beca Carter Hollings & Ferner Ltd report "Earthquake Damage Assessment – 151 Worcester St", dated 27th May, 2011 notes the following damage observed during the inspection:

- *The building has suffered from column sway damage between level 1 level 2 resulting in permanent column offset estimated to be between 20 and 30mm at the front of the building. A survey would be needed to confirm extent of the offset.*

- *Significant damage to concrete perimeter columns and beam column joints on the lower 3-4 levels indicating column hinging of the moment resisting frame. Significant damage to the interior columns and beam column joints at the lower 3 levels including extensive concrete spalling and cracking exposing reinforcing indicating column hinging of the interior frame.*
- *Cracks and localised failure to infill concrete block walls at the truck dock area.*
- *It is anticipated there may be some cracking to the concrete floors given the offset of the frame and given there was some damage to the concrete precast flooring. A detailed inspection would be required to confirm the extent of cracking of the topping slab. This would require removal of the hung ceiling system and floor finishes.*
- *Very limited inspection of the precast flooring seating due to presence of ceiling tiles. Some indication of damage to stahlton precast flooring beams at seats adjacent to interior columns.*
- *Stairs were not structurally separated between floors and have considerable damage. Stairs generally not safe to access.*
- *Broken glazing. Damage cladding system including extensive cracks and loss of parts of cladding panels on rear (north elevation). Observed damage generally concentrated around the beam column joint frame areas on the lower floors. Cracking to cladding observed at adjacent building where pounding occurred. Loss of weather tightness.*
- *Cracked plasterboard partitions including separation of interior partitions at exterior walls. Damage observed to suspended ceiling, lights and ceiling services.*
- *Pounding damage against adjacent building on west side of building. Adjacent buildings on west side tagged yellow.*

Key issues identified in the inspections were noted in the report as:

> *The vertical and lateral load support system for the building is damaged and the load carrying capacity of the building for both vertical and lateral loads is compromised.*

During our inspection of the building during demolition we noted;

- Damage to the pre-cast concrete wall cladding panels on the west wall. The panels were detailed with mid panel connections top and bottom and at mid-height between floors with a 15mm clearance all round.

  It appears that mortar and weathering sealants reduced the clearance such that the rotation induced by inter-storey lateral deflections of the building resulted in compression crushing and spalling at panel corners and excess tension at the lower connection, causing severe cracking and spalling at the base of the panels. The top connections (with a stronger detail) were relatively unaffected.

- Inelastic deformation and some hinging is noted at the lower level of the main frame rectangular columns and top and bottom of some of the circular columns at the junction with the rectangular section "capital" at each floor.

- Some vertical cracking of the column capital at level 1 at the northwest corner as a result of axial load/ compression dilation of the column capital.

No inspection of the interior of the building was possible.

Photographs of the damage to the building are included in Appendix 2.

Structural performance
It appears that the building has racked with a column sway mechanism resulting in a permanent offset of the upper structure to the west.

The damage observed internally by Beca is noted as: *plastic hinging damage to the columns and beam column joints rather than the beams.* Beca described the development of a strong column/ weak beam mechanism which is evident in the north and south walls. We understand that Beca had assessed that the building was 40 to 50 years old, and therefore designed to an earlier code (probably assumed to be the 1965 code). Beca appear not to have been aware of that the primary lateral load resisting elements were the two internal east-west frames. The damage to the pre-cast panels on the west wall was concealed by the adjoining building at the time of their inspection.

As the building was designed in 1987 the structure was presumably designed to the NZS4203: 1984 loadings code and the NZS 3101: Part 1: 1982 concrete code.

We are of the opinion that the building suffered a tortional response as a result of the horizontal eccentricity caused by the combination of the following;

- additional stiffness from apparent lack of separation of pre-cast wall panels to the west exterior wall, which were designed to rock on a central fixing, however on site were found to have the horizontal joints solid filled with mortar, and
- additional stiffness from infill block work walls at ground floor,

The extent of damage at the horizontal junction of the precast panels demonstrate that these elements were not effectively structurally isolated from the west wall frame. The panels are detailed as having a 15mm seismic separation, but this does not appear to have been provided during construction, possibly due to the use of shims and mortar. During the earthquake these joints locked up as the frame would be expected to have an inter-storey deflection of 80mm at 2.5% drift, significantly more than the movement joints could have sustained. The effect of the lack of separation is assessed as causing the centre of rigidity to shift towards the west wall and to induce a significantly greater torsional response in the building than that which would have been taken into consideration in the design of the building.

The two seismic frames in the east-west direction, were located on the internal frame lines and would therefore less effective in resisting portional forces.

The block work infill walls to level 1 (the ground floor) may have stiffened this level enough that a column sway mechanism was induced at the level above as noted by Beca.

Issues Arising from Review

Earthquake prone status

The building was identified as being earthquake prone in 2008, and the building status was noted to be a "Dangerous" building by the CCC on the 5th September 2010. We are uncertain as to the basis of the assessment and question that the correct building was assessed. We would not expect the building to be earthquake prone.

Changes in building code

The building at 151 Worcester Street was designed in 1987 and is now 24 years old, having reached approx 50% of its design life. The introduction of the Building Act 1991 and significant revision of design codes have occurred since the building was designed.

A review of the performance of the building at 151 Worcester Street in the Canterbury Earthquakes must make allowance for advances in the seismic design of structures that have occurred since the building was designed.

While the building is expected to have met the performance objectives of the design codes prior to the introduction of the Building Act 1991, a review of the building performance identified the following structural aspects that need to be enhanced to achieve resilient building performance.

Structural regularity

The building incorporates several undesirable features, namely:

- The stair and lift opening coinciding with the location of the seismic resisting frame along the west wall
- The north wall frame being irregular with only one central column on the ground floor and two internal columns on each level above ground floor.
- The presence of the precast panels along the west wall with mortar packing provided between the top of lower panel and the underside of upper panel, which stiffened the west wall frame.
- The east-west seismic frames being poorly located to resist the torsional response induced by the inadequate separation of the panels to the west wall.
- The stairs to the building were not detailed to withstand the inter-storey deflection that occurred in the Canterbury earthquake series.

The extent of damage to this building is likely to have been affected by the undesirable features incorporated into the buildings layout, in particular the inadequate separation of the precast panels along the west wall of the building.

We suggest that building codes should penalise undesirable or irregular building layout and that torsional effects should be considered after the development of inelastic behaviour in one of the lateral load resisting elements in each direction.

Seismic detailing of gravity support structure

The damage to the north and south wall gravity frames demonstrates the importance of detailing of all structural elements of a building. Although these elements were not selected by the designer as lateral load resisting elements, they were required to support the gravity loads combined with vertical acceleration effects under the deformations of the building during the buildings response to the 22nd February, 2011 earthquake.

Severe damage to these elements of the building contributed to the need to demolish the building.

It is necessary that all structural elements should be designed to retain load carrying capacity under the predicted structural deformations at the ultimate limit state.

Separation of non structural elements

The severity of shaking significantly contributed to the larger than expected inter-storey lateral deformations, which resulted in the extent of damage to the building. The lack of separations at the exterior pre-cast panels and the concrete block infill is assessed to have contributed to the damage.

The detailing provided insufficient seismic separation between the precast panels and the frame. This restricted the movement of the west wall frame which induced unfavourable modes in the response of the structure. The strong tortional response would have been substantially different than the "regular" response of the building that the designer was expecting.

Effective separation of non structural elements such as precast panels and stiff block work walls is important to achieving reliable structural performance. It is suggested that seismic separation should be provided for an adequate margin beyond the calculated ultimate limit state deformations.

Seismic detailing of stairs

There is a fundamental need to maintain safe egress from a building following a significant earthquake. This requires the stairs to be detailed to withstand the inter-storey deformations during the earthquake without damage or loss of support. It is suggested that stairs should be designed and detailed to withstand the inter-storey deformations for an adequate margin beyond the calculated deformations under an ultimate limit state earthquake. The design must provide for such movement in both the longitudinal and transverse directions.

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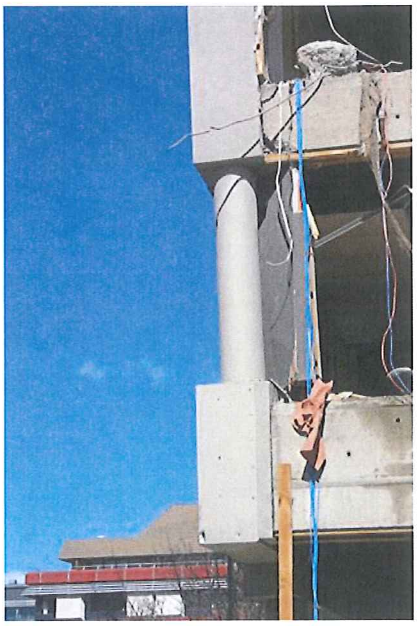
APPENDIX 1

Site Plan

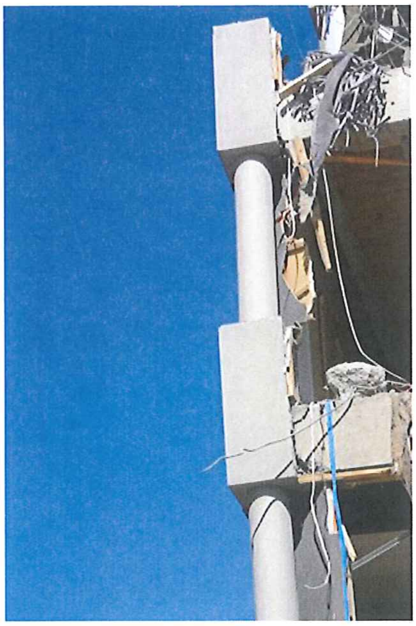

Gloucester St
NORTH
Manchester St
Mw7.1 - 4th SEP 2010
No151
Mw6.2 - 22nd FEB 2011
Latimer Square
Worceste

APPENDIX 2

Specific photographs of damage from the 22nd February 2011 earthquake.



Damage to columns at east end of north wall



Damage to columns at east end of north wall

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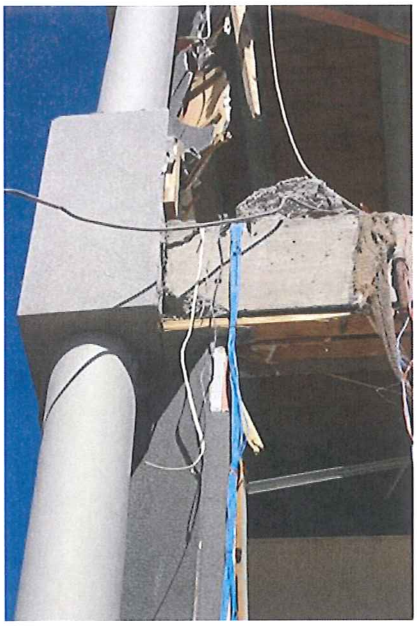
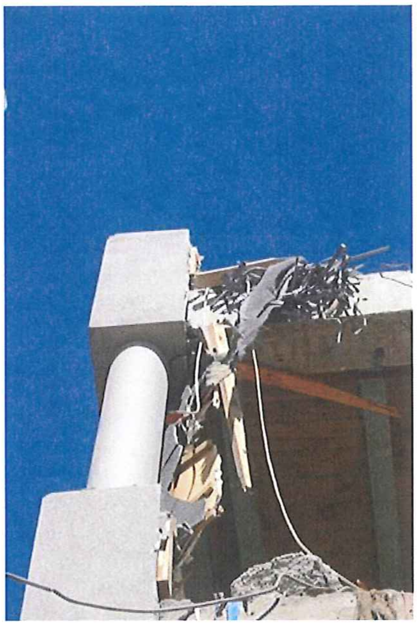


Damage east end north wall frame



Damage east end north wall frame – upper level

Damage to east end north wall – ground floor

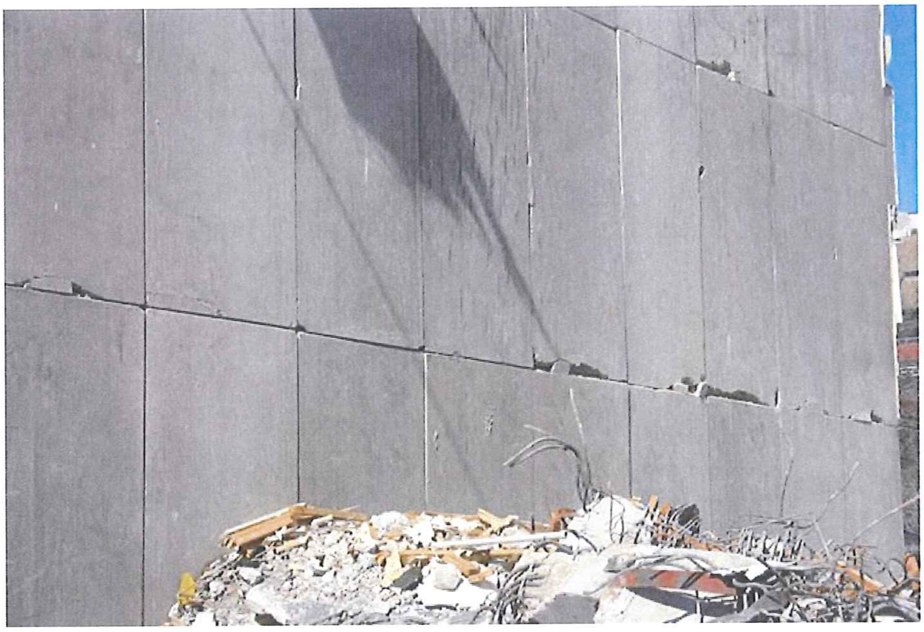
Damage west end north wall frame - upper levels

Damage east end north wall frame – lower levels

Damage to west end of north wall frame – lower levels

Damage to west end of north wall frame - upper levels

Damage to panels west wall – lower levels



Damage to panels lower level of west wall

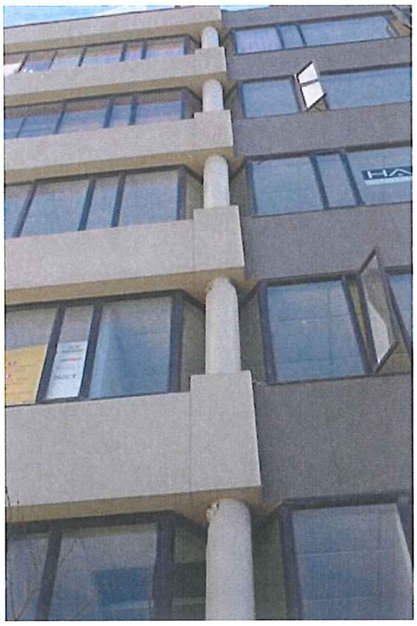
Damage to lower level of precast panels

Damage to precast panels – west wall

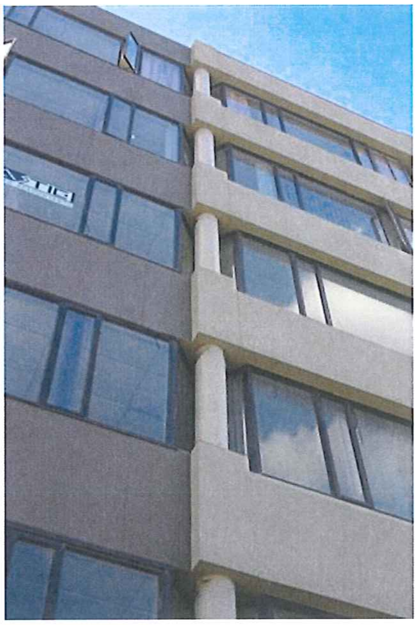
Damage to south wall frame

Damage to south wall frame

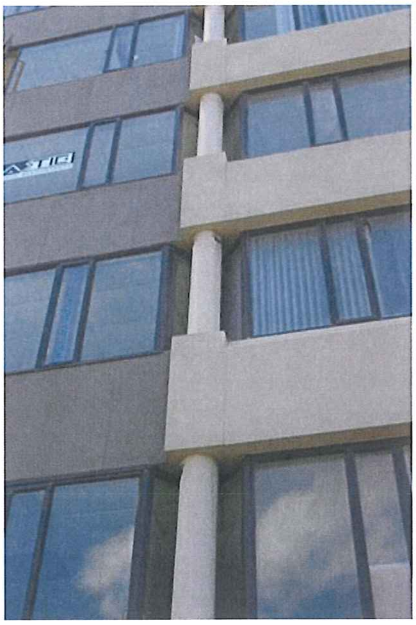
Damage to south wall frame



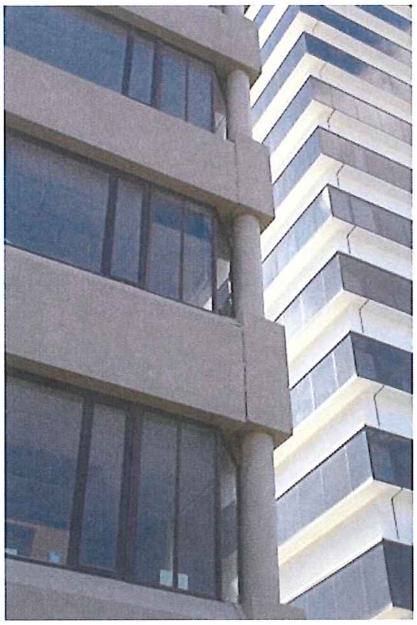
Damage to south wall frame



Damage to south wall frame



Damage to south wall frame

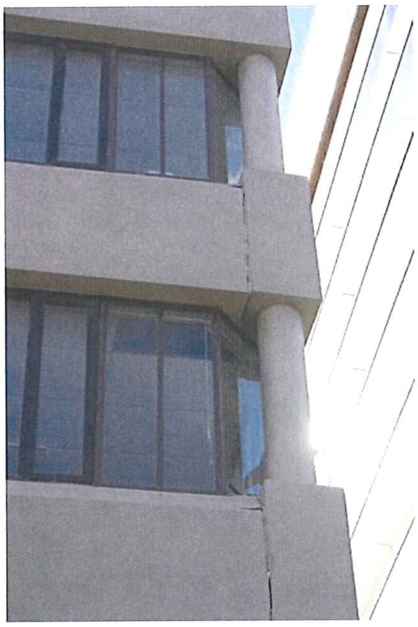


Damage to east end south wall frame

Damage to east end south wall frame

Damage to east end south wall frame

Damage to east end south wall frame



Damage to east end south wall frame