INDEPENDENT ASSESSMENT ON EARTHQUAKE PERFORMANCE OF

CRAIGS INVESTMENT PARTNERS HOUSE - 90 ARMAGH 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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February 2012

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 known as Craigs Investment Partners House at 90 Armagh 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 an inspection of the interior of the building on the 9th November 2011.

Location of Building

The building was located at 90 Armagh Street, Christchurch City, with Armagh Street to the north and Oxford Terrace to the west. To the east is the building at 100 Armagh Street, and to the south is a significant gap of approximately 1.5m to a three level building.

The Avon river is approximately 20m away from the building to the west.

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 90 Armagh Street is a 10-storey building plus basement that had been constructed circa 1986. The building is rectangular in plan oriented east-west and its structure consists of three reinforced concrete frames along the building in a east-west direction and two reinforced concrete frames in the north-south. The basement level contained car parking, the ground level had a restaurant, and the remaining floors contained office space.

The building was designed by Sheppard & Rout Architects and Holmes Wood Poole & Johnstone Limited consulting engineers in 1985. We have reviewed the engineering drawing set as provided.

The exterior cladding consists of precast concrete panels to the south and east wall, and a glazing system to the north and west sides.

Gravity System
The flooring was a "double tee" precast concrete flooring system with a 65mm insitu concrete topping slab orientated so that spans ran north-south in two bays between the three reinforced concrete frames in the east-west direction. The topping slab was reinforced with standard HRC 665 mesh. The "double tee" units were flange hung with 75mm seating onto the concrete beams.

Foundations

The basement slab is a concrete raft foundation varying between 300mm and 900mm thick. The retaining walls to the basement are 300mm thick cast insitu concrete.

Seismic System

The building was constructed with perimeter reinforced concrete moment resisting frames to each side of the building. The end columns to each frame are set approximately 2.5m back from the corner of the building such that there are no corner columns. The frame along the middle of the building is a predominantly gravity only frame. There is a diagonal beam installed to the corners of the building such that the corners are truncated on plan.

On grid A to the western face one of the columns of the frame lands on a transfer beam at level one to allow for the vehicle access to the basement.

The columns to the external frames are 900mm x 450mm reinforced concrete columns, and the beams are 320mm by 900mm deep when the concrete floor topping is included. The frame is made up of "T" sections of precast beam-column units, with the frame joined together with a cast insitu joint mid span of the bay, and with the columns spliced with a mechanical fixing denoted *type "U" NMB splices*. A typical "T" section is shown in Figure 1.

The seismic frames are 10 levels high, with the basement walls taking seismic loads below ground level, with the walls in alignment with the frames on all sides except the north side where the wall was approximately 4m north of the frame.

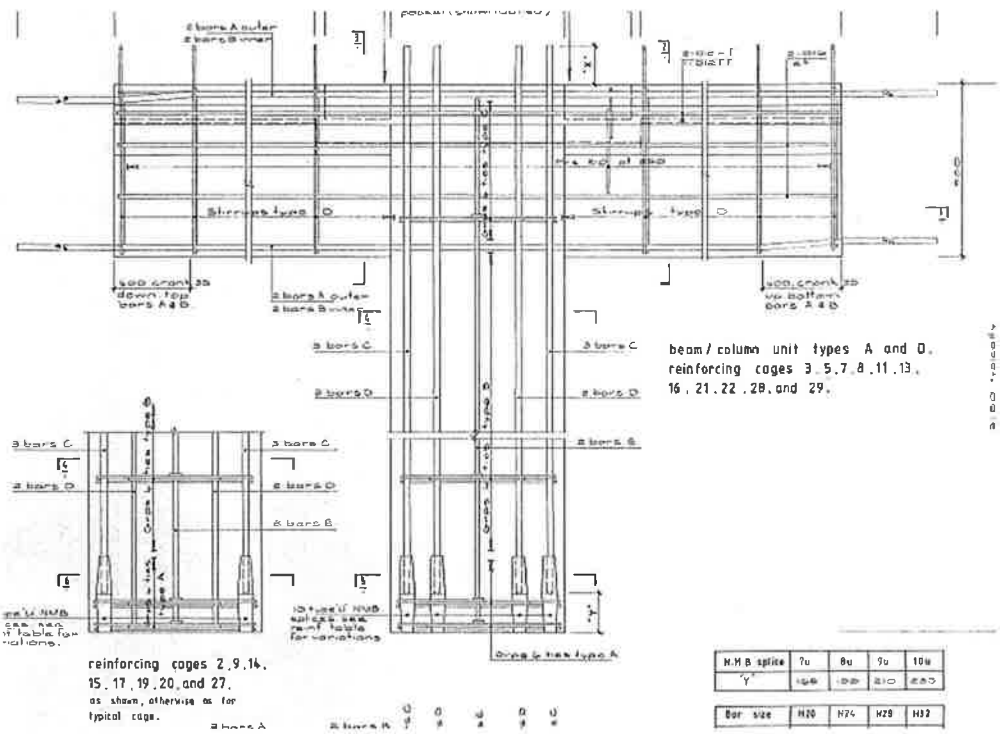

Figure 1 Elevation of precast "T" section

Floor diaphragm

Diaphragm action was provided by the "double tee" flooring system with a 65mm concrete topping slab for the majority of the floors. The ground level floor is of 250mm thick insitu slab and acts as a transfer diaphragm to transfer loads to the perimeter basement wall on the northern side.

The floor has a significant cut out for the location of the stairs and the lift shafts. This is located near the centre of the building on the southern side and extends to the centre of the building as shown in figure 2.

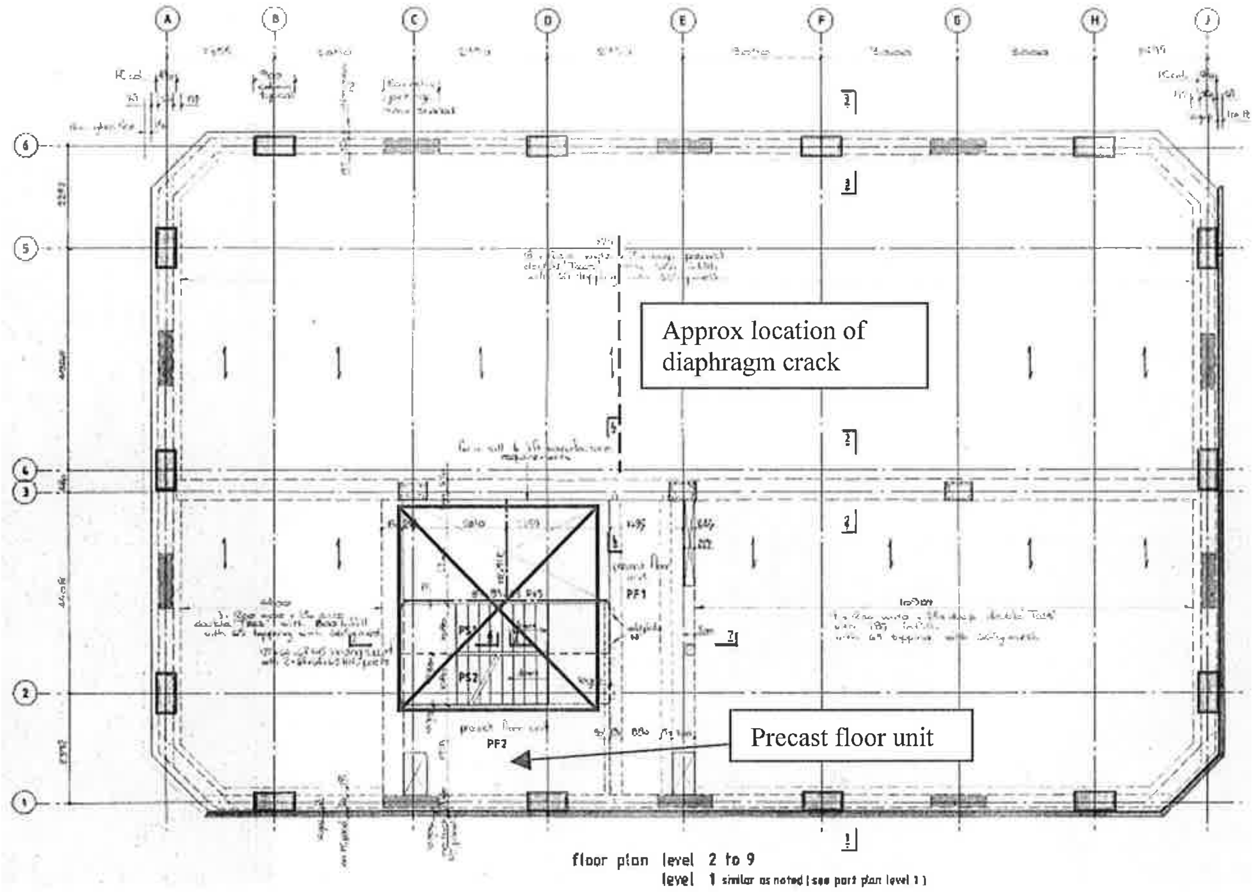

Figure 2 Slab plan showing stair and lift shaft opening

Stairs

The main stairs are constructed from precast reinforced concrete units in a dog leg stair setout, and the end of the unit adjacent to the floor level is welded to cast in plates, and the mid height landing is supported on an RHS frame which would allow some deflection and allow for movement between levels.

Precast panel walls

The precast concrete wall panels to the south and east exterior walls are 100mm thick, and are supported with two 80NB tubes cast into the top of the beam above, with two fixing brackets with slotted holes to fix to the level below to allow for lateral movement.

Compliance

The Christchurch City Council issued a building permit for the project on the 8th of September 1986. As the building was constructed prior to the introduction of the Building Act 1991, no code compliance certificate was involved.

In September 2003 a Code Compliance Certificate was issued by the Christchurch City Council for a small extension and shop front glazing to the north west corner at ground level.

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 damaged in the 22nd February, 2011 earthquake.

A Rapid Assessment Level 1 dated 25th February, 2011 by Opus International Consultants Limited assigned the building a red placard. The building was also surveyed by Aurecon to determine the tilt of the building.

However, a Rapid Assessment Level 2 dated 1st March, 2011 by Opus International Consultants Limited assigned the building a yellow 1 placard allowing limited entry.

Subsequently, a Rapid Assessment Level 2 dated 2nd March, 2011 by Powell Fenwick Consultants assigned the building a red placard and noted the existing placard as red also.

A further Level 2 assessment was undertaken on 14th March, 2011 by Opus International Consultants Limited assigned the building a yellow 1 placard allowing limited entry.

The damage noted for all of these assessments was consistent and is as per that noted in the next section.

Structural Performance

Opus International Consultants Limited report "90 Armagh Street – Craigs Investment House, Preliminary Building Assessment", dated 28th February, 2011 notes the following damage observed during the inspection:

- *The site has been subject to liquefaction and lateral spreading.*
- *The building has rotated on its foundation and now leans towards the southeast by approximately 300mm at the top of the building; a lean of approximately 0.5 degrees.*
- *The basement structure appears to have suffered little damage.*
- *The moment resisting frames have sustained plastic deformation in the beams at the columns faces, particularly in areas of the corner columns on the northeast and northwest corners. However, plastic deformation is not highly advanced, and we consider there is remaining capacity in the frame.*
- *There is also plastic deformation away from the column faces, at approximately midspan of some beams. We suspect that these areas have not been detailed as potential plastic hinge zones.*
- *The diaphragms in first to fifth floor have fractured through, with crack sizes approximately 4mm observed. There is no indication of any reinforcement continuity across these cracks, so we expect that this has fractured. We*

anticipate that diaphragms above this level have also failed, but did not continue our inspection above fifth floor level.

During our inspection of the building, in addition to the damage noted above, we noted;

- Pounding damage between the building and the building to the east at 100 Armagh Street. The damage is minor and only appears to be the glazing from the building at 90 Armagh Street that has been damaged. The movement between the buildings was significant as evidenced by the drainage pipes from 100 Armagh Street being crushed between the buildings, indicating up to 200mm relative movement at the top of the building.
- The corner precast beams that trim between the two exterior frames show cracking damage and cracks extend to the cantilever beam portion of the seismic frame.
- From levels 7 to 9 of the building a horizontal crack is apparent that is located under the beam-column joint and curves upwards across the lower area of the panel zone of the joint. This cracking appears to become prevalent in the upper levels of the building as the plastic hinge beam cracking that appears to the lower levels of the building reduces in nature.
- The diaphragm cracking extends to the level 6 and may be to the higher levels, however, upper floors are covered by floor coverings.
- No sign of damage to the precast panel walls, indicating that the seismic separation detailing was effective.
- The restraints to the water tanks to the roof were ineffective as the fixings for the steel strap were screws into concrete walls that pulled out.

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

Settlement of the building

The primary effect of the seismic events on the building has been the rotation, settlement and resulting tilting of the building. The raft foundation has rotated and settled as a consequence of the shallow founding depth of the foundation and ground liquefaction and lateral spreading damage.

The settlement to the south-east may be related to the additional building weight on those two faces from the precast wall panels, however, without geotechnical information on the underlying ground profile the comment cannot be substantiated.

It should also be noted that the seismic resisting system within the building may not have been subjected to the full effect of the seismic accelerations due to potential rocking of the building and "base isolation" from the liquefiable soils under the building.

Performance of moment resisting frame

Inspection of the moment resisting frames establishes they have performed well. There was cracking of an expected nature to the beams in the plastic hinge zone adjacent to the face of the columns. The beam cracking seen below level 7 is similar to other buildings in Christchurch, which has been of a single vertical crack mode to the face of the columns, however, some of these cracks show the expected fan of diagonal cracking.

Above level 7 the cracking to the beam-column joint transitioned to a crack to the column located at the underside of the joint, but which extended up to be located above the horizontal reinforcing steel of the beam. This was able to be inspected on the inside face of the column only.

The columns were detailed with high strength reinforcing, grade 380MPa, and the beams with mild reinforcing, grade 275MPa, and as such were of a "weak beam/ strong column" design philosophy. Therefore, the horizontal crack to the column is more likely to be a result of severe vertical acceleration in combination with the frame action.

There was damage to some of the insitu concrete mid-beam splices at the lower levels where the cover concrete to the reinforcing had spalled off the precast section adjacent to the splice.

The precast section of beam located in the corners of the building that effectively truncates the corner of the building was detailed to be located with a pin connection at each end. However this section effectively allowed the force and any effects of beam elongation from the frame to transfer around the corner. This is evident in the cracking into the cantilever section of the precast beams adjacent to the corner.

Diaphragm
The floor slab of this building was torn through the middle of the building, primarily due to the combined factors of the 65mm thickness, a lack of sufficient ductile reinforcing, the concentration of the beam elongation, and the cutout for the stairs and lift shaft that created a "neck" to the slab.

The effective diaphragm has an irregular shape due to the openings in the slab, as well as the makeup area being of a precast floor unit. This effectively reduced the depth of the diaphragm to approximately half in an area of high stress where the diaphragm acts to transfer the load between the two external seismic frames.

Detailing of precast wall
The detailing of the precast walls to the eastern and southern façade of the building performed well. The movement joint that was allowed for was for 65mm movement is either direction, and this compares favourably well with 2.5% drift for a 3.2m floor to floor height with would need 80mm movement. This is significantly more than was typically allowed for in buildings of this era.

Issues Arising from Review

The building at 90 Armagh Street was designed in 1986 and is now 26 years old, having reached approximately 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 90 Armagh 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.

Foundations

The rotation and settlement of the building was as a result of a shallow raft foundation system for a heavy building founded in an area of potentially liquefiable soils within the ground profile and potential lateral spreading.

We are unable to reach conclusions on the foundation failure without being able to review the geotechnical reports for the building or the calculations, however, it is apparent that this form of foundation system was inappropriate for the ground type at this location.

It is also important that records on the geotechnical information available at the design stage and the calculations should be received and retained by the local authority.

Moment resisting frame
The moment resisting frames to this building below level 7 exhibited the same beam cracking as other buildings in Christchurch, which was of a single vertical crack mode to the face of the columns. Above level 7 the cracking transitioned to a crack in the column located above the horizontal reinforcing steel of the beam.

The single crack is unusual in that it differs from the traditional fan of diagonal cracking that is exhibited in the slow load increment testing of specimens in a laboratory environment. The single crack, and the horizontal crack to the columns at the upper levels of the building is probably attributable to the short duration, violent nature of the earthquake of 22 February, 2011 and the significant amount of vertical acceleration.

Diaphragms
The changes in the design codes since the design of this building would appear to have adequately addressed the design issues regarding the lack of positive ductile reinforcing in the floor slabs to transfer lateral loads to the load bearing elements.

At the time of design of this building, there was a common assumption of a rigid diaphragm.

The floor slab of this building is irregular due to the openings in the floor and the use of a precast floor unit, and has a "neck" through the centre of the slab where the stresses are highest. In recent years, the importance of proper consideration for the design of floor diaphragms has been emphasised within the industry and the need to adequately design the diaphragm for the forces it is expected to transfer.

Seismic detailing of secondary elements
The importance in providing for adequate resilience in the detailing of secondary elements to withstand the inter-storey deformations under an ultimate design earthquake have been included in changes in the design codes since the design of this building.

The designers need to be aware of the importance of achieving a pin connection where a pin connection is assumed in the analysis or for considering the affect that member restraint can induce in the adjoining structure under severe earthquake.

Pounding between buildings
The pounding between the buildings at 90 and 100 Armagh Street was restricted to damage to glazing of 90 Armagh Street, and the drainage pipes to 100 Armagh Street, and there does not appear to be any "hard" clashes between the buildings.

However, the glazing damage was due to a decorative detail from the façade of 100 Armagh Street extending into the seismic gap, and the drainage pipes were also located in the seismic gap. The drainage pipes would be very difficult to repair given the limited space between the buildings. The building at 90 Armagh Street was set 120mm from the boundary to the east from the drawings, which is not a significant building as this building would deflect approximately 500mm with 1.5% drift.

It is important that buildings are located such that they do not pound against one another, and that all items of the building are excluded from the seismic gap.

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APPENDIX 1

Site Plan


NORTH
Durham St
Armagh St
Colombo St
Gloucester St
Mw7.1 – 4th SEP 2010
No.90
Mw6.3 – 22nd FEB 2011

APPENDIX 2

Specific photographs of damage from the 22nd February 2011 earthquake

Armagh Street elevation

Oxford Terrace elevation

View of building tilting to south

Separation to building to the south

Pounding damage with the building at 100 Armagh Street

Close view of pounding damage

Damage to ground adjacent to the building at 100 Armagh Street

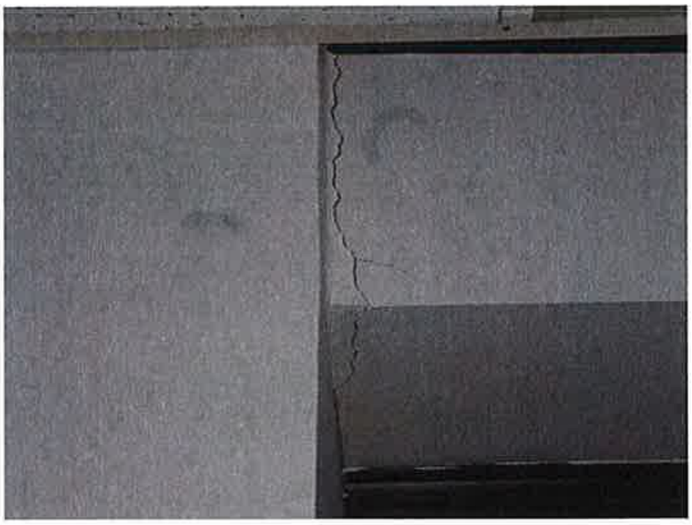
Damage to ground adjacent to the building at 100 Armagh Street

General view of splice to beam

Cracking at insitu splice to beam

Spalled concrete to insitu splice of beam

Spalled concrete to insitu splice of beam



Typical single crack to beam column joint

Typical single crack to beam column joint

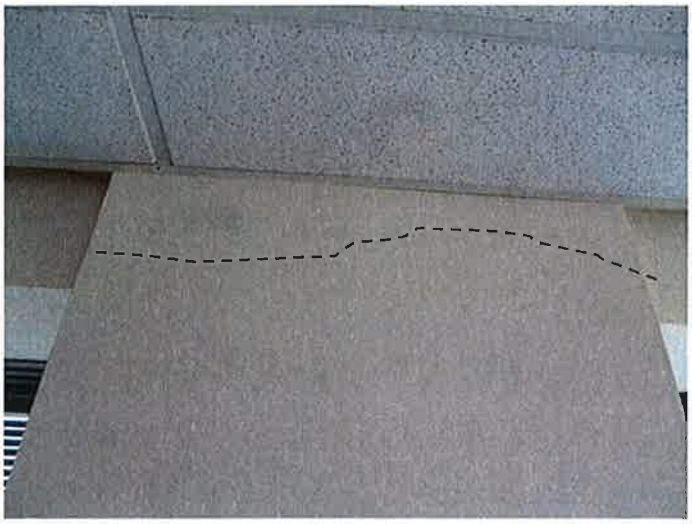
Fan cracking to beam adjacent to column

Damage to corner beam

Damage to corner beam

Floor diaphragm crack

Horizontal crack to column typical above level 7



Horizontal crack to column typical above level 7

Broken drainage pipe to building at 100 Armagh Street due to pounding

Strapping to water tank to roof loose