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INDEPENDENT ASSESSMENT ON EARTHQUAKE PERFORMANCE
OF
Bedford Row Car Park Building

FOR

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

Report prepared by Peter C Smith and Jonathan W Devine
OF
Spencer Holmes Ltd

January 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 Bedford Row Car Park Building at 20 Bedford Row, 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. An inspection of the building was undertaken before the building was demolished.

Location of Building

The building was located at 20 Bedford Row. The site extends from Bedford Road to Lichfield Street. The location of the site and the epicentre of the main earthquakes are shown on an aerial photograph of Christchurch in Appendix 1.

Description of Building

The Bedford Row Car Park building was constructed on a site between Lichfield Street and Bedford Row in central Christchurch. The site has a 35 metre frontage to Lichfield Street and Bedford Row and is 40.43 metres deep between the two streets. The car park had a total of twelve levels; the overall height of the building was equivalent to a six-storey building as the levels on the eastern and western sides of the building were off set half a storey in height. (Refer attached plan in Appendix 2). An access stair was provided centrally on the Bedford Row frontage. Access ramps for vehicles were located adjoining the egress stair at the Bedford Row end and the street frontage at the Lichfield Street end of the building.

Gravity System

Each car park level was constructed of 500mm deep Double Tee pre-stressed pre-cast concrete floor units with a 65mm topping spanning approximately 17m. A corbel on the east and west walls supported the pre-cast Double Tee floor units and the floors were supported by a pre-cast reinforced concrete beam supported by a central row of columns along the centreline of the building. The foundations to the building were piled under the east and west walls and shallow spread footings under the central row of supporting columns. The east and west walls of the building were constructed as 200 thick concrete tilt up panels to a height of 9.27 metres. A further 6.9 metres of 200mm thick block work was constructed above each of the 4.035 m wide tilt up panels. A 100mm wide corbel was provided to the tilt up panel at each level to support the Double Tee floor units. A corbel was cast into the blockwork walls in the upper levels to provide support for the Double Tee flooring units..

The central columns were pre-cast with rebates provided below each floor level to support the pre-cast beam, which in turn supported the Double Tee floor units.

Seismic System
The primary lateral load resistance in the longitudinal direction was provided by the inplane action of the 4.035 m wide tilt up panels with the concrete masonry vertical extension along the east and west walls. In the transverse direction, lateral load resistance was provided by cast in-situ walls along the Lichfield Street frontage and between the stairs and the ramps at the Bedford Row frontage. These walls were both of 240mm thick cast in-situ reinforced concrete construction.

The ramps were constructed integrally with the floors at each level and were used as the element to transfer lateral loads from the floor to the east-west shear walls at the north and south ends of the building.

Secondary Elements
The stairs at the Bedford Row frontage were of reinforced concrete construction. The stairs were formed as pre-cast elements, which were tied into the structure of the building using cast in-situ landings.

Compliance

A building consent for the building was issued by the Christchurch City Council on 30th October 1987. The only compliance documentation for the building that we have received were Lewis and Barrows plans 2984-1 to 22. These documents were the official building approved documents as they are stamped by the Christchurch City Council as approved subject to the bylaws dated 30th October 1987.

As the building was constructed prior to the introduction of the Building Act 1991 there was no requirement for the building to receive a code compliance certificate.

We have assumed that the building was design in accordance with NZS 4203 : 1976.

The building appears to have complied with the Building Act 1991 due to the building having been constructed prior to the introduction of the Building Act 1991 and the building having not been altered or subjected to a change of use since the introduction of the Building Act 1991.

Events Subsequent to 4th September 2010 Earthquake

There is no record of damage to the building as a result of the 4th September, 2010 or the 26th December, 2010 earthquakes. The building was damaged in the 22nd February 2011 earthquake. The building was inspected by Stuctex and the Structex report dated 21st July 2011 identified the following damage to the building.

Level Three Floor
The level three floor partially collapsed midway along the building. Structex attributed the floor failure to the failure of a corbel supporting a main central beam.

East and West Walls
Diagonal shear cracking and sliding shear cracking in the pre-cast concrete walls which generally act as shear walls resisting earthquake loading.

Double Tee Floor System
Extensive spalling to the webs of the Double Tee floor units which seat on the continuous concrete supporting corbels cast in the east and west walls.

Concrete Ramps
Severe fracturing and spalling of the concrete ramps at most levels. This was particularly evident adjacent to the north and south shear walls.

Concrete spandrels
Most concrete spandrel connections were cracked and in some cases severely spalled, significantly reducing the spandrel support.

Connection of Pre-cast Panels to Floor Plate
Spalling of reinforcing steel slab ties in the floor at the connection between the pre-cast panels and the floor plate.

Structex concluded that the structural damage to the building was extensive and that it was, in their opinion, not practicable to be repaired.

No level survey of the building was undertaken to establish the extent if any, of differential settlement. Structex comment that no liquefaction was observed on the site and no gross settlement of the building was observed. This was confirmed in our inspection.

Structural Performance

The failure of the Level 3 slab is assessed as having been caused by the rotation of the pre-cast beam supporting the Double Tee flooring units on the supporting corbel. The support for the pre-cast beams was formed by creating a rebate in the central column. The depth of the rebate was the depth of the pre-cast beam plus the depth of the Double Tee flooring units. The rebate alternated on either side of the central column to suit the off-set floor levels.

The pre-cast beams that supported the Double Tee floor were formed with a vertical duct at the support locations and this duct was threaded over a vertical D24 rod projecting from the column corbel as the pre-cast beam was installed.

The Double Tee flooring units were then placed on the pre-cast beam and extended approximately 900 beyond the pre-cast beam. There was no connection between the pre-cast Double Tee flooring system and the pre-cast supporting beam. There was also no connection between the 65mm thick topping reinforced with 664 mesh and the central columns.

Under earthquake loading in the east-west direction, the lateral load from each floor and the east and west walls was transferred through diaphragm action to the insitu concrete shear walls via the inclined ramps which interconnect each level of the building.

Under the relatively severe shaking which occurred on the 22nd February, 2011 earthquake, there was damage to the connection of the inclined ramp to the insitu shear walls and at the base of the inclined ramp where the horizontal shears within each floor diaphragm transferred into in-plane loads in the inclined ramps. The transfer of the seismic shear from the diaphragm through the inclined ramps induced vertical forces into the supporting frame at the end of the ramp. The change in the direction of the structural element resisting the seismic shear resulted in damage to the floor system at the juncture of the ramp and the floor disphragms.

It is assessed that there was sufficient translation of the third floor diaphragm relative to the central column to cause rotation of the pre-cast beam supporting the Double Tee flooring. By rotating the pre-cast beam, the load on the corbel supporting the pre-cast beam increased through wedging of the floor and pre-cast beam in the rebate in the central columns, there being no direct load transfer mechanism between the central columns and the floors. The increased vertical load resulted in the failure of the corbel supporting the pre-cast beam over the central portion of Level 3.

We are of the opinion that had a gap been left between the top of the floor toping to the Level 3 slab and the underside of the recess in the central columns for the Level 3 slab, the movement would have occurred without inducing a wedging action which significantly increased the load on the corbel supporting the pre-cast beam.

A feature of the damage to the building was the connection between the pre-cast panel/blockwork cantilever shear walls along the eastern and west walls of the building and the floor diaphragm.

Deformation of the cantilever shear elements under the intensity of shaking experienced on the 22nd February, 2011, resulted in small but significant vertical displacements at the end of each panel. As the panels were continuously connected to the floor diaphragm, the deformation of the cantilever shear walls resulted in significant local distortion of the floor at the junction between panels. This structural incompatibility has resulted in damage to the pre-cast panel / blockwork connection and in some cases to the Double Tee flooring units.

Code Changes Since 1987

The building was designed in 1987 and is assumed to have been designed to the then current loading code NZS4203 1976.

The basic design lateral load coefficient for the building under NZS 4203: 1976 was 0.11g. This compares with a lateral load coefficient of 0.10g under NZS 1170.5: 2004 of the current loadings code. This comparison assumes a period of 1.0 second, limited ductile behaviour and deep site sub-soils.

There are also significant changes to requirements for detailing of ductile and nominally ductile structural elements.

Issues Arising From Review

Severity of ground shaking

Recorded ground accelerations in central Christchurch during the 22nd February, 2011 earthquake were in the order of 0.9g.

It is evident that the building did not have the ductility to withstand the high ground accelerations that occurred in the 22nd February, 2011 earthquake.

Floor failure

Failure of the third floor along the central column line was assessed to be due to wedging of the third floor and supporting beams in a restrictive recess in the central column.

As the floor was not connected to the central column, deformations of the building under the 22nd February, 2011 earthquake resulted in rotations of the supporting beam. These rotations induced excessive load into the corbel supporting the 3rd floor and caused failure of the corbel.

It is recommended that structural components not required as part of the lateral load resisting system should be capable of accepting the deformations of the lateral load resisting system with a margin at the ultimate limit state without inducing secondary forces in the structure.

Floor connection to pre-cast panel / blockwork shear walls

The design concept for the east and west wall was that each individual pre-cast panel / blockwork wall acted independently. These panels were not connected together at the junction between panels to develop a single shear resisting wall. As a result, seismically induced deformations of the shear walls resulted in a structural incompatibility at the floor support at the joint between individual panels.

This structural incompatibility caused damage to the anchorage of panel starters in the topping to the slab, damage to both the corbel and the web of the Double Tee flooring units.

It is necessary that the design and detailing of primary structure should provide allowance for deformations which occur in all components of the structure under the ultimate limit state design earthquake.

Diaphragms

The floor diaphragms to the building lacked a continuous chord member along the east and west walls which were constructed of multiple pre-cast concrete panels/blockwork elements. The diaphragms were notched at the ramps each end of the building and were not provided with an inadequate tension chord at the edges of the diaphragm and at the notch for the ramps. Further, the induced shears within the diaphragm, were required to be transferred into inclined ramps in order that the shear could then be transferred into the shear walls at the north and south ends of the building.

The load transfer mechanism was complex and damage was caused to the junction of the ramps and the diaphragm due to the abrupt change in the direction of the structural elements transferring the seismic shears into the lateral load resisting shear wall element. Further, while no damage occurred within the floor diaphragm at the joints between the pre-cast concrete panels/blockwork walls on the east and west walls, we do not consider that the detailing of the diaphragm at the junction between the panels was sufficiently robust to provide a reliable diaphragm for the transfer of seismic shears in the east west direction.

Cracking of the pre-cast concrete panels/blockwork shear walls to east and west walls

Our interpretation of the reinforcement in the pre-cast concrete panels at the lower level is 4 – D16 at each side of the panel. Calculations indicate that the tensile strength of the concrete may have exceeded the tensile capacity of the reinforcement at the ends of these units. It is likely that the single cracking which has occurred near the base of the units has caused strain hardening of the reinforcing steel at these locations as the tension capacity of the reinforcing steel is insufficient to promote more distributed cracking at the base of these panels.

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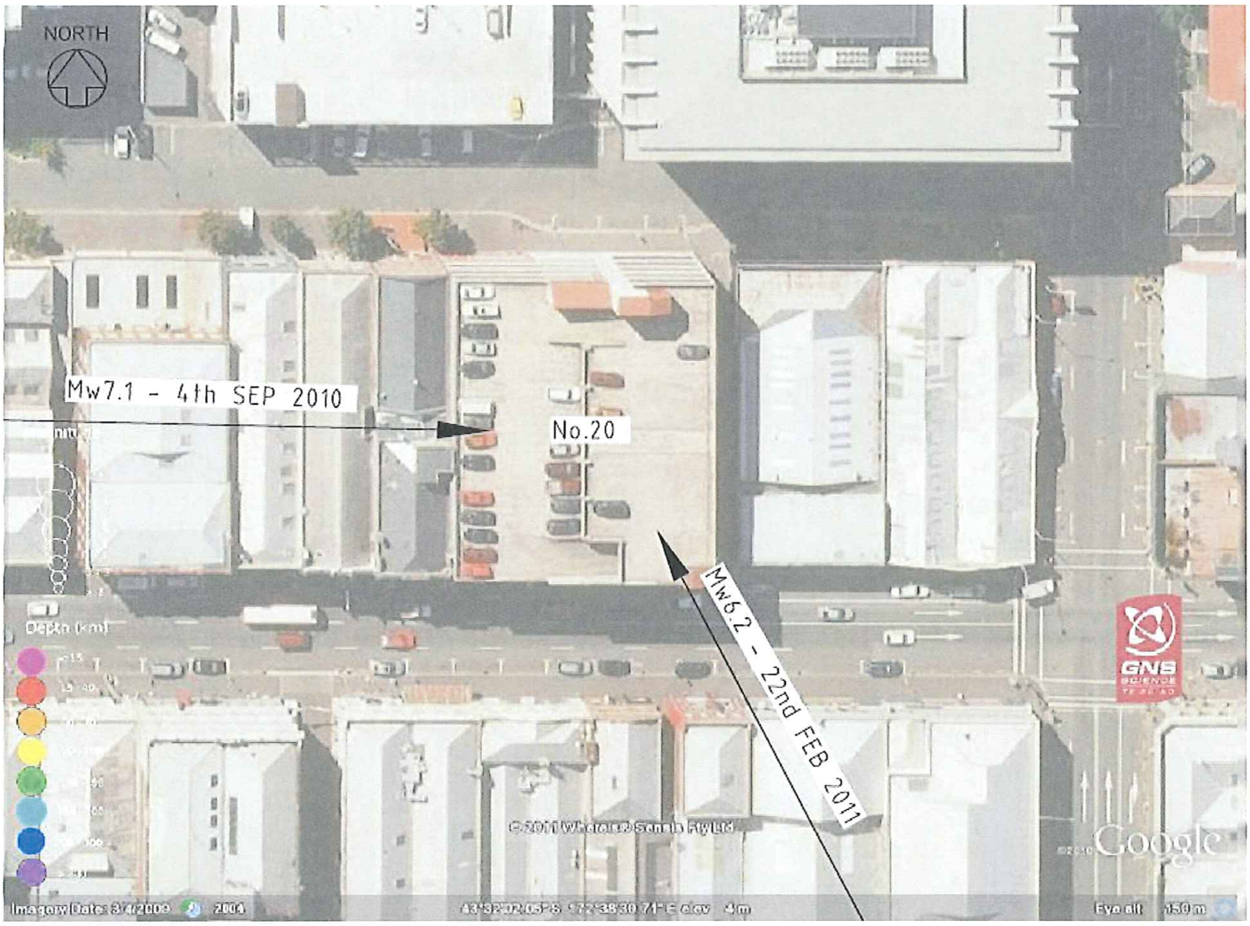
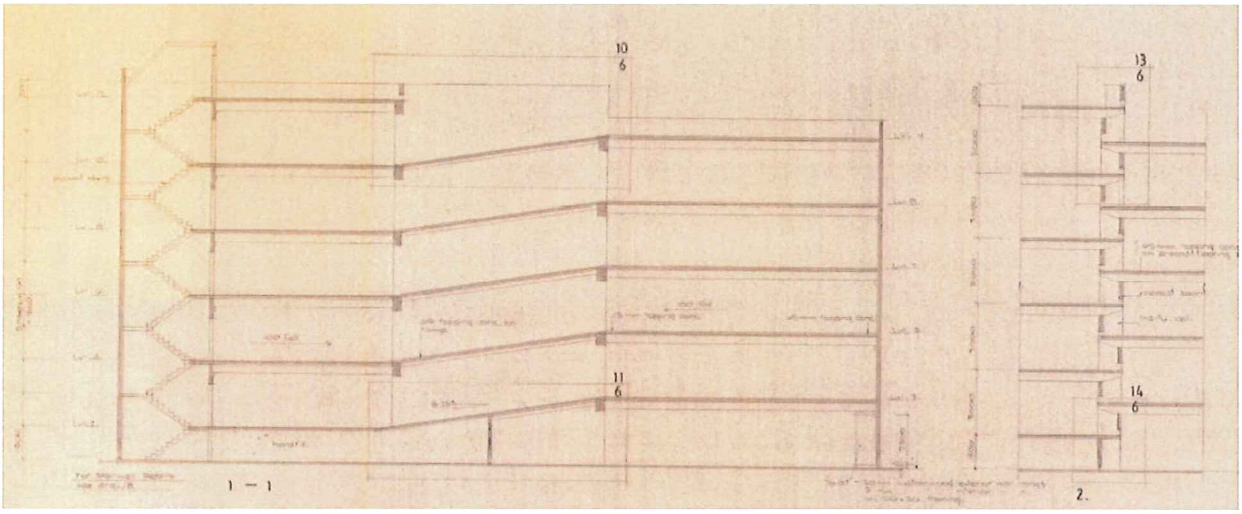
APPENDIX 1:
Location of Building


NORTH
Mw7.1 - 4th SEP 2010
No.20
Mw6.2 - 22nd FEB 2011
Depth (km)
GNS SCIENCE
Google


Bedford Row, Christchurch Central, New Zealand
Google

APPENDIX 2:
Typical Floor Plan


10
6
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2.

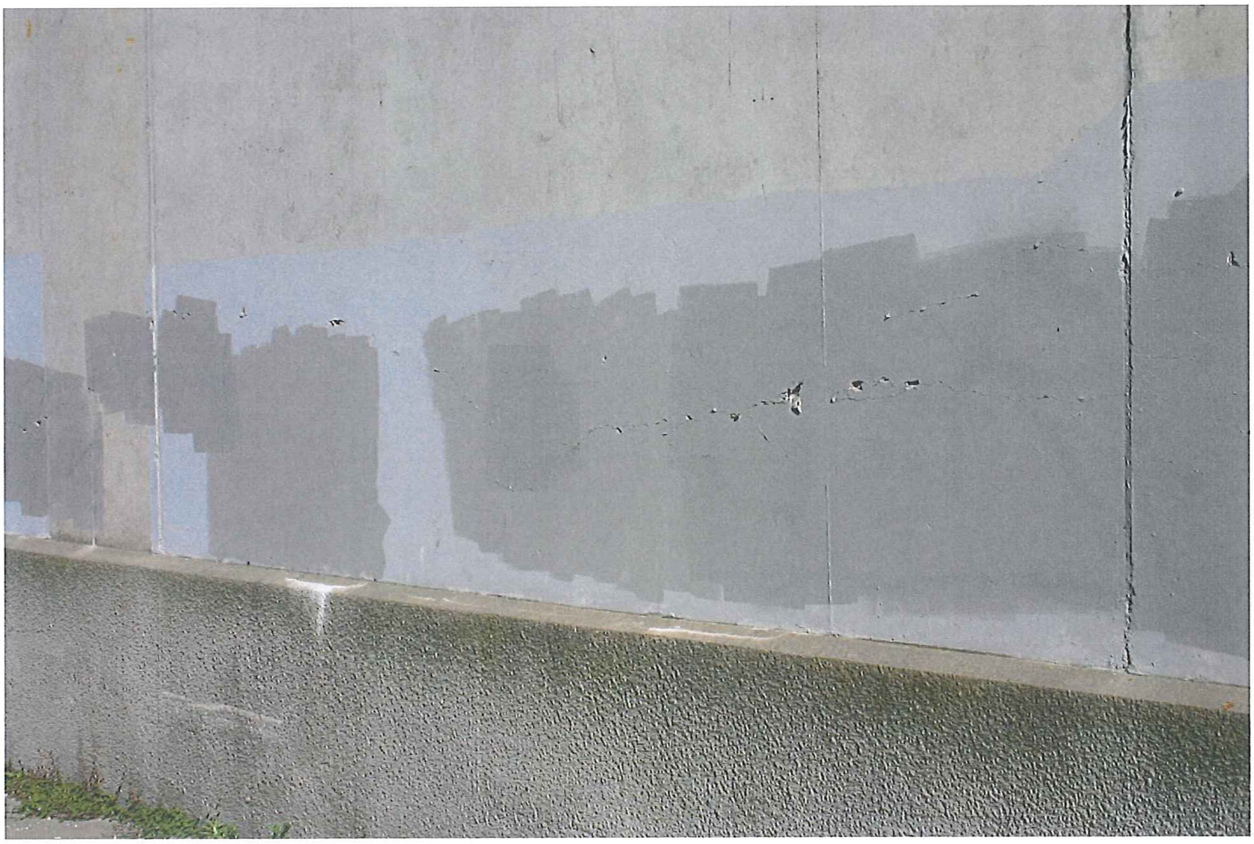
APPENDIX 3

Damage following the 22nd February, 2011 Earthquake

Failure of level 3 floor, viewed from level 3

Failure of level 3 floor, viewed from level 2

Damage to Double Tee flooring units



Damage to east wall panels

Damage of ramps

Damage at ramps