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Report

Investigation into the Collapse of the Pyne Gould Corporation Building on 22nd February 2011

Prepared for Department of Building and Housing (DBH)

By Beca Carter Hollings & Ferner Ltd (Beca)

26th September 2011

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on behalf of	Beca Carter Hollings & Ferner Ltd		

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Executive Summary

The New Zealand Department of Building and Housing (DBH) has commissioned Beca Carter Hollings & Ferner Ltd (Beca) to undertake an investigation into why the Pyne Gould Corporation (PGC) Building at 231-233 Cambridge Terrace collapsed during the Magnitude 6.3 earthquake that struck Christchurch at 12.51 pm on Tuesday 22nd February 2011.

This report has been prepared under the direction of a panel appointed by DBH to oversee investigations into four buildings damaged or collapsed in this earthquake.

The five-storey building, designed in 1963, is founded on shallow pads, and its lateral resilience was provided by walls surrounding the stairs and lifts. These walls form a core, and were approximately symmetrically located around the north-south centre line of the building but offset from the east west axis. The axes of the rectangular building are orientated approximately north-south/east-west. These walls had openings in them in some areas.

Our calculations confirm that these core walls were reinforced to meet the seismic design loadings current in 1963 (NZSS 95).

A significant assessment of the building's earthquake resilience was undertaken in 1997 for the new owner. This identified significant shortfalls in resilience with respect to the loadings standard current at that time (NZS 4203: 1992).

During a 1998 major refurbishment, additional (steel) columns were added to the perimeter reinforced concrete columns to enhance their vertical load-carrying capacity. Some investigations were undertaken into providing additional horizontal load resilience via steel bracing but no additional horizontal resistance was added. Some openings in the concrete walls were infilled and others created. The decorative reinforced umbrellas on the roof were taken down because they were considered seismically unsafe.

In 2008, further changes were made to an opening in the central walls, and a 12-metre steel telecommunications mast was added to the central core walls above the Roof Level.

No significant structural damage was observed after the 4th September 2010 earthquake, although there was some non-structural damage. Similarly, no significant structural damage was recorded after the Boxing Day (26th December 2010) Magnitude 4.9 earthquake.

Witnesses have advised of damage observed after the 4th September earthquake. Some of this, but not all, has been correlated with known spalling from reinforcing bar corrosion and recorded damage. The photographs seen of this damage confirm that it was minor and would not have provided warning of the collapse that was to occur.

In the 22nd February earthquake, the building's collapse eastwards appears to have been initiated by the failure in compression of the eastern core wall between Levels One and Two. Almost no structural damage was observed between Ground Level, and Level One. The core walls above Level Two were reportedly largely undamaged. The east half of the roof detached itself from the core and slid partially off the level below on to the adjacent building.

Analytical models of the total structure and of the core walls alone have been created. Non-linear time-history analyses using actual records of the three earthquakes have been undertaken.

Concrete and reinforcing steel tests of elements of the collapsed building do not indicate strengths or characteristics less than those expected at the time of design.

Soils investigations, additional to those for neighbouring sites for other building developments over the life of the building, have been undertaken at the site and at the nearest earthquake recording site (REHS). It has been concluded that there has not been any deformation of the site that would be instrumental in the collapse of the structure. This is confirmed by the site survey that has been completed. The building site appears to be somewhat stiffer than the REHS site. It is possible that the intensity and frequency content of the shaking at the site for the three earthquakes was less than that recorded at the REHS but, on balance, the records obtained from this site are considered to be the most appropriate for investigation of the collapse of this building.

Our analyses all confirm that the core wall between Level One and Level Two had insufficient capacity, by a considerable margin, to resist the intensity and characteristics of the ground shaking recorded at the nearest instruments on 22nd February 2011.

The performance of this building during the 22nd February earthquake has highlighted the potential vulnerability in large earthquakes of lightly, centrally-reinforced shear-walls with flanges and without concrete confinement, especially where the horizontal resistance to earthquake is provided solely by the shear-wall.

It is recommended that any guidelines for the assessment of buildings be reviewed to confirm that buildings of the Pyne Gould Corporation Building type (i.e., lightly, centrally-reinforced shear-walls where horizontal seismic resistance is provided solely by the shear-walls) will be identified as potentially poorly-performing in earthquakes and, if necessary, the guidelines should be revised.

Further investigation of the seismic performance of existing lightly reinforced shear-walls is considered a priority.

In our opinion:

Original Design

- The structure when built met the 1963 design requirements of that time for the prescribed earthquake loads, both in terms of the level of strength and the level of detailing provided.
- Testing of concrete and reinforcing steel from some elements after the collapse did not indicate that they were less strong than required by the design.

Modifications

- Modifications made to structural elements (addition of perimeter steel props and insertion/deletion of doorways in the core walls) during the life of the building were not material with respect to the collapse on 22nd February 2011.

Comparison with Current Code

- Pre-September 2010, the building achieved between 30 and 40%*NBS* (new building standard) when assessed against the New Zealand Society for Earthquake Engineering Guideline recommendations.

Damage prior to 22nd February 2011

- Damage to the structure was observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes to the:
  - tops and bottoms of the perimeter columns
  - core walls (cracking)
  - stairs (cracking).
- This damage was relatively minor and not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking.

- The proposed method of repair at that time of grouting the cracks appeared reasonable.

Mode of Collapse

- The building collapsed when the east and west reinforced concrete walls of the core between Level One and Level Two failed during the earthquake.
- The west wall yielded in vertical tension, and then the east wall failed catastrophically in vertical compression.
- The ground floor structure stayed intact and virtually undamaged as it was significantly stronger and stiffer than the structure above.
- Torsional response (i.e., twisting of the building about a vertical axis) was not a significant factor.
- Once the west wall had failed, the horizontal deflections to the east increased markedly.
- The perimeter columns and/or joints between the columns and the beams, and the connections between the floor slabs and the shear-core, failed consequentially at some levels, causing the floors to pancake.

Reasons for Collapse

- The damage observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes did not significantly weaken the structure with respect to the mode of collapse on 22nd February 2011.
- The shaking experienced in the east-west direction was almost certainly several times more intense than the capacity of the structure to resist it.
- The connections between the floors and the shear-core, and between the perimeter beams and columns were not required at the time of design to take, nor were capable of taking, the distortions associated with the core collapse.

Commentary

- Neither foundation instability nor liquefaction was a factor in the collapse.
- Extensive studies undertaken in 1997 for a previous owner confirmed that the structure was below the current standard at that time with respect to earthquake resilience for new buildings.
- The capacity of the building in 1997, after the addition of the steel props behind the perimeter columns, was judged, at that time, to be in excess of 50% of the then current new building standard.
- The current owner's structural engineers inspected the building after the 4th September and 26th December earthquakes and advised the owner that it was acceptable to occupy it.

1 Introduction

The New Zealand Department of Building and Housing (DBH) has commissioned Beca Carter Hollings & Ferner Ltd (Beca) to undertake an investigation into why the Pyne Gould Corporation (PGC) Building at 231-233 Cambridge Terrace collapsed during the Magnitude 6.3 earthquake that struck Christchurch at 12.51 pm on Tuesday 22nd February 2011.

This report has been prepared under the direction and review of an expert panel appointed by DBH to oversee investigations into four buildings damaged or collapsed in this earthquake.

2 Objective and Scope

The following are the objectives and scope set for this investigation by the Department of Building and Housing:

The purpose of this technical investigation into the performance of buildings in the Christchurch CBD in the 22nd February Christchurch is to establish and report on, for specified buildings:

- *The original design and construction of the buildings*
- *The impact of any alterations to the buildings*
- *How the buildings performed in the 4th September 2010 earthquake, in particular the impact of the earthquake on the building*
- *What assessments - including the issuing of green stickers and any further structural assessments - were made about the buildings' stability / safety following the 4th September 2010 earthquake*
- *Why these buildings collapsed or suffered serious damage*

The investigation will take into consideration:

- *The design codes, construction methods, and building controls in force at the time the buildings were designed and constructed and changes over time as they applied to these buildings*
- *Knowledge of seismic hazard and ground conditions when these buildings were designed*
- *Changes over time to knowledge in these areas*
- *Any policies or requirements of any agency to upgrade the structural performance of the buildings*

The investigation will use records of building design and construction, and will also obtain and invite evidence in the form of photographs, video recordings and first-hand accounts of the state, or the performance, of the buildings prior to, during, and after the 22nd February 2011 aftershock.

Matters outside the scope of the investigation

The investigation and report is to establish, where possible, the cause or causes of building failures. It is not intended to address issues of culpability or liability arising from the collapse of the building. These matters are outside the scope of the investigation.

For the PGC Building the scope of the investigation has included consideration of the following:

- Interviews of eye witnesses to the collapse and rescue activities following.
- Structural analyses.
- Materials testing.
- Geotechnical investigations.
- Site surveys.

3 Approach/Methodology

3.1 General

At the commencement of our investigation, the demolition of the PGC building had taken place down to Level One for the western half, and to Ground Level on the eastern side. Most of the debris had already been taken to the Burwood Landfill. Concrete and reinforcing steel samples had already been taken from columns and beams (but not the core walls), and demolition was proceeding rapidly. We were able to look at the remaining ground floor structure (virtually undamaged), and immediately requested that demolition not proceed beyond the Ground Floor level so that the foundations would be preserved. This was approximately two months after the collapse.

At this time, there was no definite visible evidence that the site had been deformed vertically or had moved sideways with respect to surrounding features.

The DBH advertised publicly for those with observations they wished to be considered by the investigators to make these available.

As there was no seismograph at the PGC site, it is not possible to be sure of the intensity or characteristics of the shaking experienced by the building in any of the major earthquakes it experienced. We therefore determined that we should obtain the records from the nearest sites, and compare the ground conditions of which they were recorded with those of the PGC site. We commissioned boreholes to be drilled through the foundation of the PGC building, and at the site of the nearest seismograph.

Witnesses to the actual collapse were interviewed with an objective of trying to determine the sequence and timing of the collapse.

While it would have been quite easy to determine the maximum earthquake strength of the building by relatively crude methods, we determined that we should also test these against other techniques which would simulate the response of the building in each earthquake, and possibly point to which elements had been highly stressed in any of them. This would also raise our confidence in our findings if we could simulate the observed sequence of collapse, and match the evidence obtainable from photos and observations by others of the collapsed building.

The most sophisticated analytical tool available in these circumstances is non-linear time-history analysis. Computer simulation of this sort has been available for around 40 years, and Beca has used this type of analysis for almost all that time. It involves setting up a theoretical model of the building which includes the stiffness and strength characteristics of all the parts of the building, including its interaction with the ground. The mass/weight of the building structure and the furniture, etc., inside it are also modelled. The earthquake records are applied to this model at approximately 1/100th of a second intervals, and the reaction (internal forces and movement) of the building computed. When parts of the building reach their capacity, the consequential loss of further resistance is modelled, and the analysis continues.

The sensitivity of the many assumptions that are required to be made can be tested by undertaking multiple analyses.

We received full co-operation from all public authorities and related private parties in obtaining documentation of the history of the building.

3.2 Information Gathering

The following data was available to us:

- An apparently complete set of structural drawings dated 1963.
- The Christchurch City Council's Property File (1978- August 2010).
- Owner's structural engineer's reports and site notes from 1997 to 2011.
- Soils information (historical) from the Christchurch City Council's borehole database (called *Orbit*).
- Soils investigations undertaken in June 2011 for the site and the REHS seismograph site.
- Lateral spreading transepts from University of Canterbury/Tonkin & Taylor.
- Photographs from many sources.
- Eye witness accounts of the collapse.
- Post-collapse test results for steel reinforcing bar and concrete.
- Testing of recently-identified concrete and reinforcing from shear/core walls.
- Site survey completed in July 2011.

3.3 Reporting

Our report covers all aspects of our investigation, and is designed to meet the information needs of both the public and peers. We have placed the more technical parts of our analyses in the appendices.

It has been reviewed by the DBH Expert Panel, and their comments addressed.

We have referenced, but not appended, the report on materials investigation commenced prior to the start of our investigation by Hyland Consultants Ltd.

Where we have directly quoted from others, we have italicised the quotation. At the request of DBH, names of companies and authors have been removed from most reproduced material.

4 Building Description

4.1 Outline/Summary

The five-storey building, designed in 1963, was founded on shallow pads, and its lateral resilience was provided by walls surrounding the stairs and lifts. These walls form a core (shear-core), and were approximately symmetrically located around the north-south centre line and offset from the east-west centre line of the building. These walls had openings in them in some areas. The axes of the rectangular building are orientated approximately north-south/east-west.

Outside the extent of the core wall the cast insitu, 6” (152 mm) thick floor slabs were supported on a grillage of reinforced concrete beams and reinforced concrete columns arranged around the building perimeter and two reinforced columns inside the building.

The perimeter columns above Level One were supported on the ends of beams cantilevering out past the line of the Ground Level columns. Refer to Figure 4.1.

Figure 4.1 : 2010 Photo of Building taken from South-East

The horizontal stiffness and strength of the building structure from Ground Level to Level One was significantly greater than above Level One by virtue of wing walls extending from the core and a significantly more robust reinforced concrete frame below Level One. Figures 4.2, 4.3 and 4.4 show, respectively, the Ground Level and Level One plans and a cross-section through the building perimeter that have been inferred from the construction drawings.

In 2008, further changes were made to an opening in the central walls, and a 12-metre steel telecommunications mast was added to the central core walls above the Roof Level.

The core-wall reinforcement shown on the drawings (this has not been able to be confirmed visually on site or from the rubble) is typically a single layer of deformed 5/8" (16 mm) rods at 15" (381 mm) centres, arranged both horizontally and vertically, and located central to the wall thickness. Laps are detailed immediately above the floor slabs.

The perimeter reinforced concrete columns above Level One were 10" square (254 mm x 254 mm) and reinforced with four and eight deformed 1" (25.4 mm) diameter vertical bars, depending on location, with typically two undeformed ¼" (6.4 mm) diameter column tie sets at 9" (229 mm) centres. There were no ties within the beam/column joint region, and the outer column bars were located outside the perimeter spandrel beam bars. These columns do not contribute significantly to the building's horizontal stability in an earthquake.

A significant assessment of the building's earthquake resilience was undertaken in 1997 for the new owner. This identified significant shortfalls in resilience with respect to the loadings standard current at that time (NZS 4203: 1992).

A full set of construction drawings and various cut-away views of the building and shear-core are provided in Appendix A1.1.

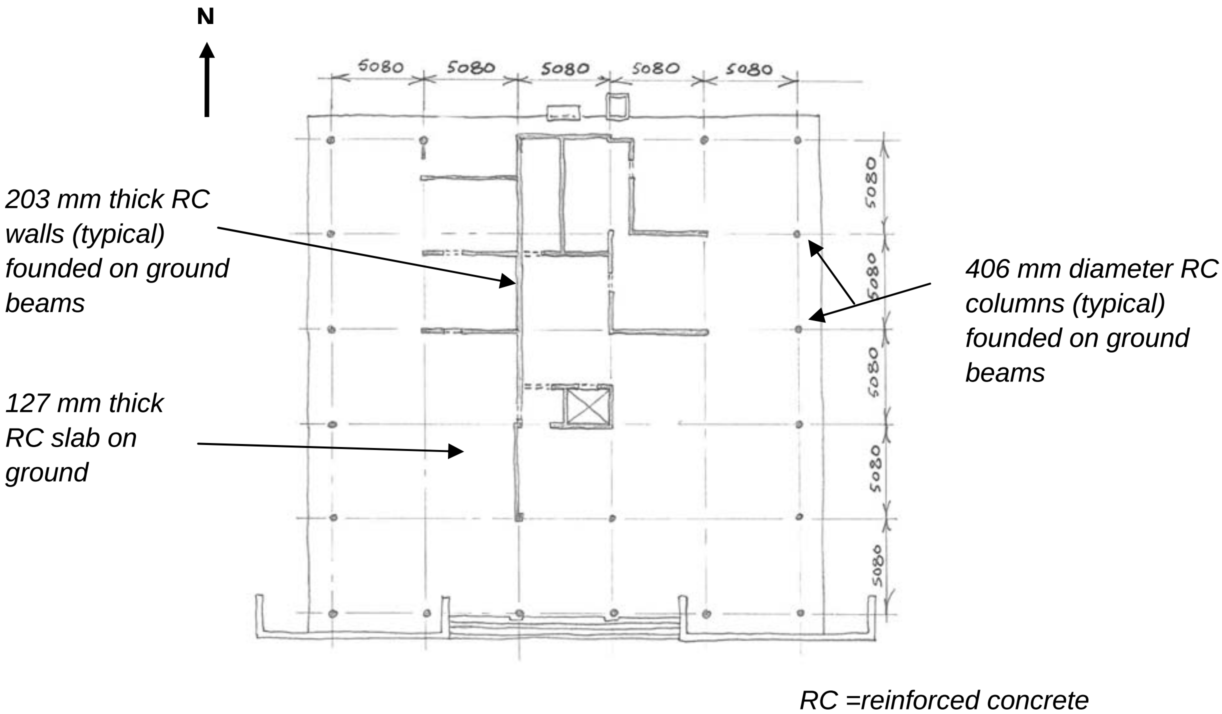

Figure 4.2 : Ground Level Plan of Building

Further views of the building and shear-core are provided in Appendix A1.3. These also show the various changes to the building since construction.

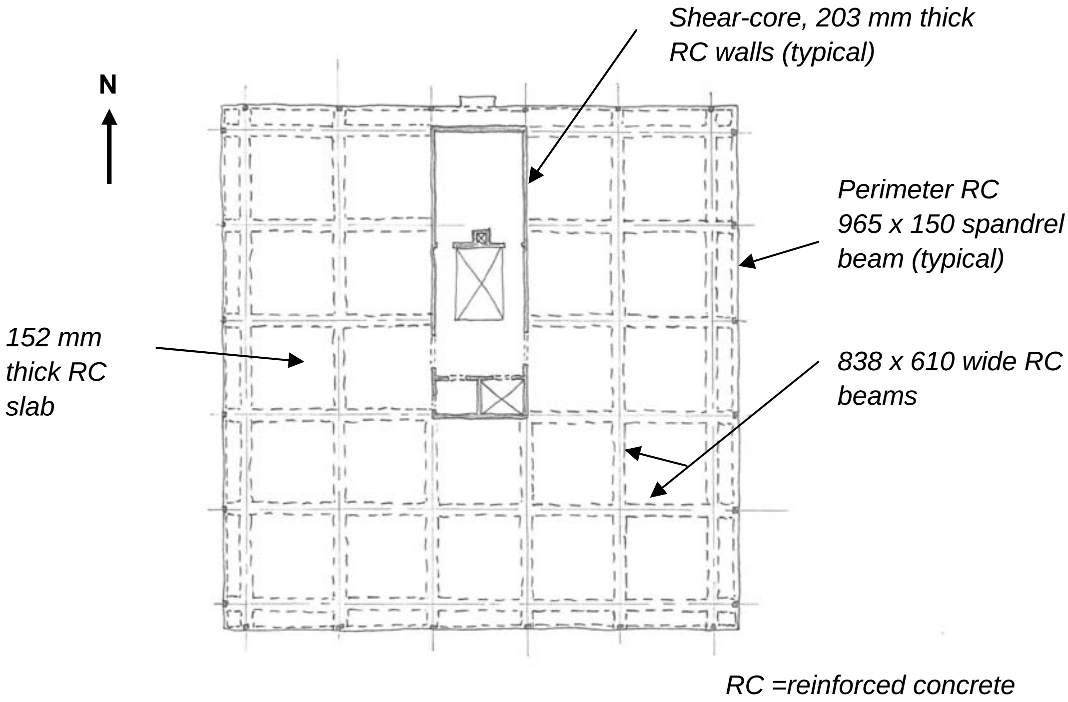

Figure 4.3 : Level One Plan of Building

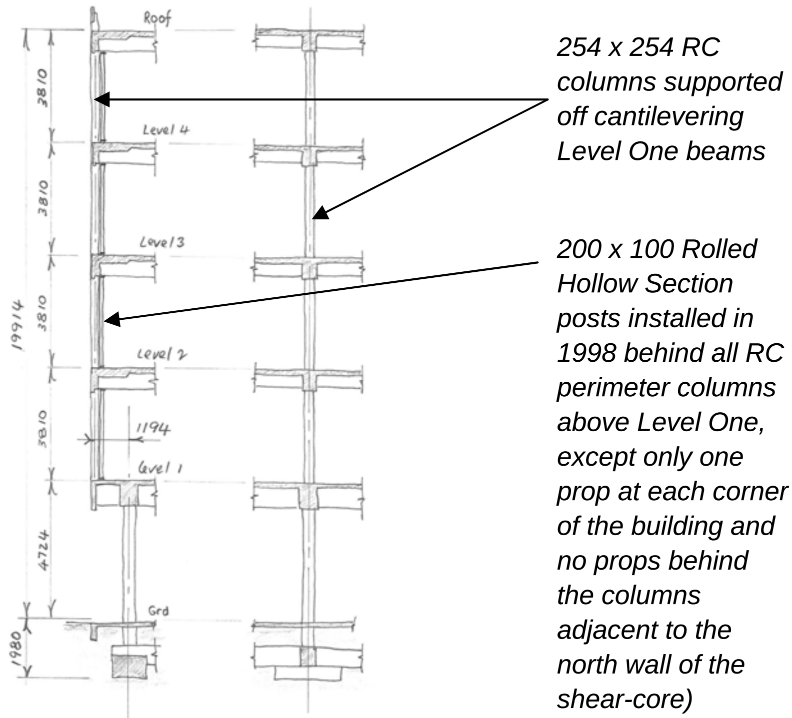

Figure 4.4 : Section through Building Perimeter

4.2 Site Investigations (Soils, Seismology)

4.2.1 Building Location

The building was located at 233 Cambridge Terrace, Christchurch. This is between Colombo and Manchester Streets on the north bank of the Avon River. Figure 4.5 shows the building and the general locale.

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Figure 4.5 : Location of Building Site

4.2.2 Information Sources

Soils

Any site-specific investigations that might have been undertaken at the time of design have not been sighted.

The Christchurch City Council has provided summary borelogs from its database (*Orbit*) for historic investigations made in the centre and in the vicinity of the site. It is understood that this database includes records from the Christchurch Drainage Board's records once published in book form.

To supplement these, and to look for any evidence of liquefaction beneath the building, cone penetrometer tests to 4-6 metres, machine boreholes to 15 metres, and cores through the ground floor slab were undertaken in June 2011 at the site which had been cleared to ground-floor level. Boreholes were also taken at the site of the nearest seismograph at the Resthaven Rest Home in Colombo Street near Bealey Avenue. These investigations are documented in a factual geotechnical report that is provided in Appendix A6.2.

Seismology

The nearest permanent seismograph to the PGC building is at the Resthaven Rest Home (REHS) in Colombo Street, about 100 metres south of Bealey Avenue. This is about 670 metres to the north north-west of the PGC building site. The next closest permanent seismographs were in the

Botanic Gardens (CBGS, 1.54 km SW), near the Christchurch Hospital (CHHC, 1.24 km SW), and near the Catholic Cathedral College in Barbadoes Street (CCCC, 1.32 km SE). Temporary seismographs were installed in the Christchurch Police Station after the 4th September earthquake. The locations of these sites relative to the Pyne Gould Corporation building site are shown in Figure 4.6.

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Figure 4.6 : Location of Building Site Relative to Strong-Motion Recording Sites

The acceleration records for the 4th September 2010, 26th December 2010 and 22nd February 2011 main shocks and many aftershocks are available from the GeoNet ftp site (internet). The majority of these, but not all, have been filtered and corrected. Information on the soil conditions beneath each station was sought from GNS Science (operators of GeoNet) in order to see whether they were similar to those underneath the PGC building. The softness and layers of the soil beneath a seismograph and a building may have a significant impact on the intensity and frequency content of the shaking experienced by them.

Permanent Ground Movement

The preliminary results of horizontal displacement transepts in the vicinity of Cambridge Terrace were made available via the Panel. It is understood that these were undertaken by a team led by Associate Professor Cubrinovski of the University of Canterbury in order to plot the extent of lateral spreading that has occurred in the vicinity of the Avon River after the 22nd February earthquake.

In early July 2011, a topographical survey of the building site and the adjoining areas was undertaken to determine whether there has been any significant change of level or horizontal displacement of the ground surface recently. A plot showing the results of this survey is given in Appendix A6.1.

4.2.3 Interpr etations

Soils

From the Christchurch City Council Database (Orbit):

- Bore 16102 was located beneath the central core of the building and indicated a 4.3 metres thick surface layer of sand and gravel overlying 6.1 metres of "wood, sand, peat lenses".
- Bore 14469 was located in the rear (northern side) car park and indicated a greater depth of gravel and sand (9.1 metres) overlying "wood pieces, sand with lenses of silt". In April 2011, there was little surface evidence of liquefaction immediately around the PGC building or beneath the Ernst and Young building immediately to the north-west.
- Further north (away from the river), bore 5666 was in Kilmore Street adjacent to the Markham Accountants building. The bore identified fine to medium sand to a depth of 18 metres, and artesian water pressures. In April 2011, extensive liquefaction and ground settlement was evident in this location, as can be expected from the borelog description.
- Bore 2093's location appears from the database to have been in the river due south of the PGC building. This location seems unlikely, as the log describes the location as Manchester Street and Oxford Terrace. It indicates an interbedded profile of sand, clay, silt and peat. Artesian water pressures are noted. In April 2011, there was minor liquefaction ejecta and lateral spreading evident in the vicinity of the band rotunda between Cambridge Terrace and the river.

From Investigations at Site Undertaken by Beca in June 2011:

- Three cone penetration tests (CPTs) to refusal at depths of between four and six metres.
- Three machine boreholes to a depth of 15 metres, with standard penetration testing at regular intervals, close to the CPTs.
- Six cored holes through the Ground Level slab.

The site is underlain by dense sandy gravels to a depth of around 10 metres beneath the centre of the building. These gravels are underlain by medium-dense sands. The gravel layer is slightly thinner beyond the rear (north side) of the building, extending to a depth of eight metres. To the front (south), the upper four metres is dominated by sand rather than gravel. The upper dense, predominantly sandy, gravel layer is considerably thicker than indicated on the Environment Canterbury (ECan, Canterbury's Regional Council) bore logs. The "wood" and "peat lenses" reported on the ECan logs were not encountered to the same degree. Wood fragments and organic inclusions were only observed in the sands below a depth of approximately 11 metres.

The cores through the Ground Level slab found little to no evidence of gaps having developed.

An analysis of the CPT 101 profile and the SPT profile from borehole BH101 put down in the centre of the building footprint indicates that liquefaction is unlikely to occur in the upper sandy gravels in peak ground accelerations (PGA) up to and beyond 0.8 g. Liquefaction of the underlying sands is indicated as being able to commence at PGAs of around 0.3 g. The thickness of the upper sandy gravels is sufficient to have prevented the surface expression of liquefaction during the 22nd February 2011 event.

Foundation/Subgrade Parameters for Structural Analysis:

The PGC building is supported on a grillage of beams and pads at around two metres below ground floor level. These vary in width from 0.9 metres to 1.8 metres in an east-west direction, with narrower (0.45 metres wide) tie beams running north-south. These foundations sit in the dense sandy gravels. They are calculated to have a geotechnical ultimate bearing capacity of over

1000 kPa under vertical loading. This is not expected to have been affected by any liquefaction occurring eight metres or more below the under-side of the foundations.

Soils investigations, additional to those for neighbouring sites for other building developments over the life of the building, have been undertaken at the site and at the nearest earthquake recording site (REHS). It has been concluded that there has not been any deformation of the site that would be instrumental in the collapse of the structure. This is confirmed by the site survey that has been completed. Refer Appendix A6.1.

Spring stiffnesses under these foundations for use in structural analysis depend on the magnitude of loading, and the following points on a curve are considered appropriate:

- 100 kPa applied load – 1 mm deflection.
- 200 kPa applied load – 4 mm deflection.
- 500 kPa applied load – 12 mm deflection.

Strong-Motion Recording Sites:

There was little definitive soils information available for these sites, and the soil profiles were initially inferred from the information collected from the Christchurch City Council's Orbit database.

CHHC – well logs of bores near the CHHC site, including bore 8542, indicate silty sandy "pug" (assumed to be clayey in behaviour) from a shallow depth to 13 to 21 metres. The pug is underlain by sand or sandy gravels. The profile is therefore significantly different to that at the PGC site.

REHS - bores 2140 to 2142 directly across Colombo St from this site indicate near-surface sands to a depth of around two metres, overlying around six metres of peat or clayey soils over sand and gravel. This profile is also significantly different to that at the PGC building site. Our investigations (June, 2011) at the REHS site comprised a machine bore to a depth of 15 metres with standard penetration testing at regular intervals, and a cone penetration test to 20 metres. The investigations identified a near-surface gravel layer extending to a depth of 1.3 metres, which is underlain by typically firm and commonly organic silt to around nine metres. Characteristic shear strengths of the silt (derived from the CPT results) range from 10-15 kPa in a two metres thick soft peat/organic silt at a depth of around five metres to a more typical 50 kPa. The silt is underlain by medium dense, becoming dense, sand. The near-surface site response is likely to have been modified by the silt layer and, in particular, the soft organic zone.

CCCC – the well log of bore 2123 near this site indicates clay and sand to a depth of 22 metres, with no closer description of any specific subdivisions. As with the REHS site, the near-surface clayey deposits are likely to have modified the site response compared with that at the PGC building where sands and gravels dominate.

GNS Science has also assessed the ground conditions at these strong-motion seismograph sites using the largest-scale published geological map (Brown and Weeber 1992), the "Black" 1856 map of vegetation and waterways, their proximity to areas of liquefaction in September 2010 and February 2011, and SPAC (Spatial Autocorrelation, a micro-tremor technique) at two of the sites.

GNS Science reported to us: "*Estimated shear wave velocity profiles and site natural period have been estimated from the Brown and Weeber 1992 geological model correlated with shear wave velocity measurements and estimates in similar materials in the lower Hutt Valley, and SPAC shear wave velocity determinations for the upper layer at CBGS and CCCC.*

The sites are underlain by between >20 and <30 m of postglacial sediments comprising marginal marine sand and silt, and gravel-filled channels (Christchurch and Springston Formations) with loess and swamp deposits in places. Underlying the Postglacial sediments are predominantly dense

Pleistocene age interglacial gravels interbedded with thinner layers of glacial soils. At about 300m depth Pliocene age terrestrial and marginal marine sediments (sand, silt, clay, peat and shell lenses, wood) overlie the basaltic rocks of the Miocene age Banks Peninsula volcanics, which in turn overlie about 400 m of early Tertiary sediments (sandstone, siltstone, conglomerate and coal measures) on Torlesse (greywacke) at about 1200 to 1500 m depth.

All (four) sites are at least Class D, deep soils in terms of NZS 1170.5, and those that experienced liquefaction would have to be classified as E if the softest soils are more than 10 m thick. It would be premature to classify areas of liquefaction as E because very thin layers that liquefied are not necessarily very damaging.

Using estimated shear wave velocities combined with SPAC measurements for the surface layer at two sites, the natural period at all these sites is more than 3 seconds, but note this estimate has been undertaken blind, without examining the records.

*Subsurface conditions are very similar at all (four) sites, but they can be differentiated on the basis of whether or not Postglacial gravel is present near the surface, or whether or not liquefaction occurred at or close to a site in either or both earthquakes.***"**

With respect to the Botanic Gardens site CBGS, GNS Science says:

"A channel of post-glacial gravel passes through the site at shallow depth, and there is more than 10 m thickness of gravel in the top 21 m of Postglacial sediments, gravel is inferred to be less than 2 m below the surface on the basis of gravel being mapped within 1 m of the surface close by to the NW and SE of the site. The "Black" map is ambiguous at this site, but seems to indicate tussock with wetland to the NE on the other side of the Avon from the site.

Liquefaction flooding and sand boils were visible after the September 2010 earthquake both to the NE and S of the site, and much more extensive and closer to the site after the February 2011 earthquake. The absence of liquefaction at the site itself suggests that there is a near-surface gravel layer. The interpretation from the SPAC results suggests this site should have been subject to liquefaction (surface layer V_s < 200 m/s) but if the boundary between potential liquefaction and no liquefaction is placed at V_s = 175 m/s, the result would be better."

The building site appears to be somewhat stiffer than the REHS site. It is possible that the intensity and frequency content of the shaking at the site for the three earthquakes was less than that recorded at REHS but, on balance, the records obtained from this site are considered to be the most appropriate for investigation of the collapse of this building.

4.3 Design, Drawings and Specifications

An apparently full set of structural drawings (20) is dated 1963. Neither design calculations nor a specification have been located.

The seismic loadings applicable in 1963 were specified by Part IV of the New Zealand Standard Specification NZSS 95, but this was superseded in July 1964 by Chapter 8 of NZSS 1900 published in July 1964, and revised in December 1965. During this period, it is understood that each territorial authority would independently resolve to adopt such a revised “New Zealand Standard Model Building Bylaw”. If later practices are indicative, it could be expected that a structural engineer would have been acquainted with the proposed changes to NZSS 95 at the time of the design of this building, and could have incorporated them. Fenwick and MacRae suggest that New Zealand universities were teaching use of the British concrete design Code of Practice CP114 (1957) at the time, and it is likely that it was also being used by practitioners.

It is also possible that the Ultimate Limit State provisions set out in ACI 318-63 were adopted as these were also being used by practitioners at the time.

4.4 Variations during Construction

No indications have been found to suggest that the as-built structure was significantly different to that shown on the structural engineer’s 1963 drawings. It appears that construction did not commence for about three years after the structural drawings were signed by the structural engineer.

4.5 Post-Occu pancy Alterations

Table 4.1 summarises the major events in the building structure’s life.

Following a structural evaluation by the owner’s structural engineer in 1997, 72 steel props were installed inboard of most of the perimeter columns from the first elevated floor to the Roof Level. The contractor’s proposal letter in 1997 also quotes for the removal of the concrete block walls forming the strongroom on the first floor and the plan safes on the first and third floors. Later fit-out sketches indicate that this demolition was undertaken. It appears that the building was stripped back to a bare concrete building at this time, and new glazing, internal walls, partitioning, ceiling and an air-conditioning system were installed. Several additional penetrations were made in the shear-core. The most critical of these penetrations, from the point of view of the likely effect on the collapse of this building, was an additional door penetration in the west wall of the shear-core at Level One. The likely influence of this penetration on the collapse of the building is discussed later in this report. The work at this time also included the filling of a large floor (stair) penetration in the Level One slab.

In 2008, extensive alterations were undertaken to the ground floor – including the cutting of two entrances through the reinforced concrete core walls, together with some thickening of the surrounding walls. On the first floor, some doorways in the same walls were filled in with reinforced concrete.

Also in 2008, a 12-metre-high cell-phone tower was added to the core wall above the Roof Level.

In 2009, some drilling of small-diameter holes (around 75 mm, possibly) through the floors was undertaken to allow the connection of services for the installation of further air-conditioning.

In April 2009, repairing of cracks in the concrete columns on the perimeter columns above the first floor level was undertaken, and was reportedly at the sites of previous repairs, and caused by corrosion.

Table 4.1 : Major Events in Building's Life

Date	Event	Comment
1963	Designed as offices for Christchurch Drainage Board	
1966	Constructed	Building Consent 1964
1989	Christchurch City Council (CCC) took over ownership	
8th Oct 1996	CCC Land Information Memorandum (LIM) states existing owner to be Christchurch Drainage Board	LIM states hazards to be a) 6750 litre underground tank flammable liquid. b) Peat and wood
May 1997	CCC Project Information Memorandum (PIM) states that "Council's records indicate the site has suspect bearing capacity due to the presence of peat. (See attached bore log profile)."	
5th March 1997	Sold by Christchurch City Council	Unoccupied except for a single tenancy on first floor
1997	Structural Report by owner's engineer	With respect to NZS 4203
1997	CCC confirmed that proposed refurbishment did not constitute a *Change of Use*.	Thus, no legal requirement to strengthen building seismically
1997-1998	Strengthening & refurbishment	Steel props adjacent to columns added. Additional penetrations added to shear-core, slab penetrations filled.
2006 Major	refurbishment	
July 2007	Owner's engineer reported on options to add lightweight floor or strengthen building with an adjacent new structure	Not proceeded with.
2008	New mobile phone site added. Additional openings made in ground floor walls.	
2008	Sold to current owner	
2009	Additional mobile phone cabinets and panels added	
2009	Repairs made to cracks in perimeter columns.	
Sept. 2010	Site Report by owner's engineer	
Jan. 2011	Site Report by owner's engineer	

4.6 Effects of Time (Settlement, Corrosion, etc.)

From observations made of the ground floor slab after all building debris/rubble had been removed, including the results of a level survey, and interpretations made of the soils under the building, it is considered unlikely that the building had suffered significant settlements since construction.

Corrosion of reinforcement in the exterior frame had been reported and remedial works carried out on several occasions. It would appear that the corrosion was limited to non-critical aspects. It is considered unlikely that this influenced the collapse of the building.

In 2009, sagging of a floor adjacent to file storage was investigated. It was concluded at that time that the then existing file storage on the slab could remain in place. It is considered unlikely that this additional floor loading, in only one part of the building, was sufficient to contribute to the collapse of the building.

5 Earthquake Effects on Site and Building

5.1 Earthquake Records

5.1.1 Nearby Strong-Motion Records (GeoNet and Canterbury Network)

The nearest (corrected) recordings of the three earthquakes have been downloaded from the GeoNet ftp site. They are at:

- Botanical Gardens. (CBGS, 1.54 km to SW)
- Cathedral College. (CCCC, 1.32 km to SE)
- Christchurch Hospital. (CHHC, 1.24 km to SW)
- Resthaven Home, Colombo Street North. (REHS, (670 m to NNW)

Except for the CCCC site, all the axes of the instruments are very close to north-south and east-west (as are the axes of the four buildings being investigated by DBH). GNS Science has re-computed the CCCC recordings to make equivalent north-south and east-west components.

5.1.2 Acceleration vs Displacement Spectra

Response spectra are a convenient way of showing the maximum force and movement that a building would experience in a particular earthquake. Every earthquake has its own signature frequencies, and every building has its own frequencies of vibration. A response spectrum shows how much the characteristics of the earthquake excite a particular building in a particular direction.

For most earthquakes, it happens to be that the predominant frequencies are in the 1-2 cycles per second range. Problematically, this is also the same frequency range of the natural shaking modes of most buildings shorter than about 5-10 storeys. The degree of resonance or amplification that is experienced by the building in a particular earthquake depends on the degree of alignment of these two ranges.

Structural engineers traditionally use a spectrum in which the earthquake-induced force (measured as an acceleration) is plotted against the building's natural period (inverse of frequency) as in Figures 5.1 and 5.2 below for spectral acceleration and spectral displacement respectively.

An even more informative display of the same information can be produced by plotting the acceleration response vs the displacement. Beca has computed the 5 % damped horizontal and vertical acceleration-vs-displacement response spectra for the four sites – with a view to using these as one estimate of building displacements and an indication of the relative horizontal movements between storeys of the PGC building. These spectra for the 4th September 2010 earthquake are shown in Figures 5.3.and 5.5.

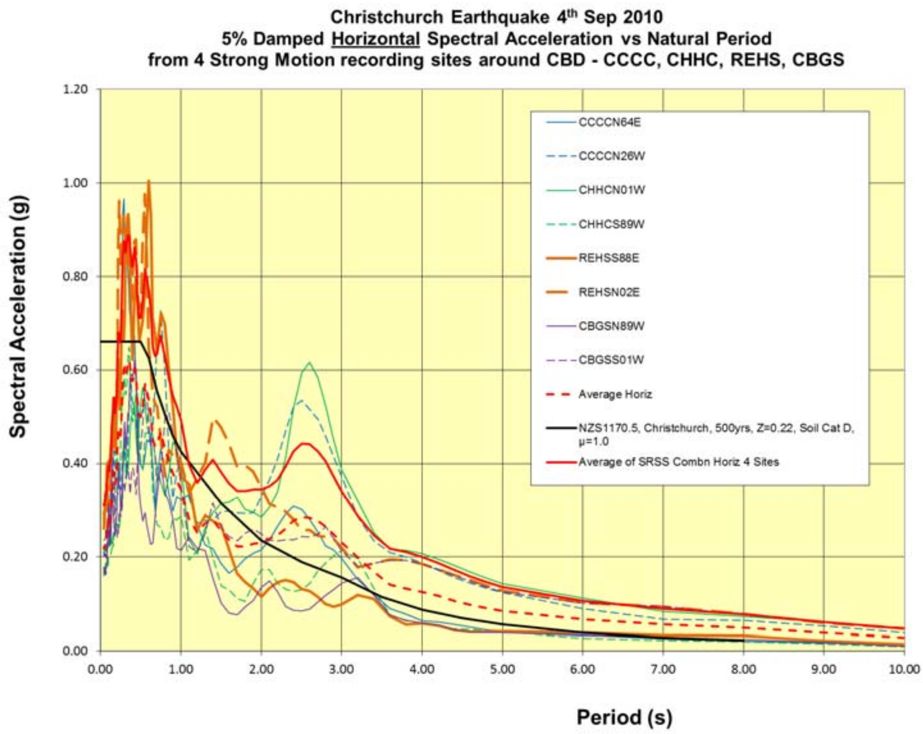


Figure 5.1 : Traditional Acceleration Response Spectra for 4th September 2010 Earthquake at REHS Recording Site (5 % Damping)

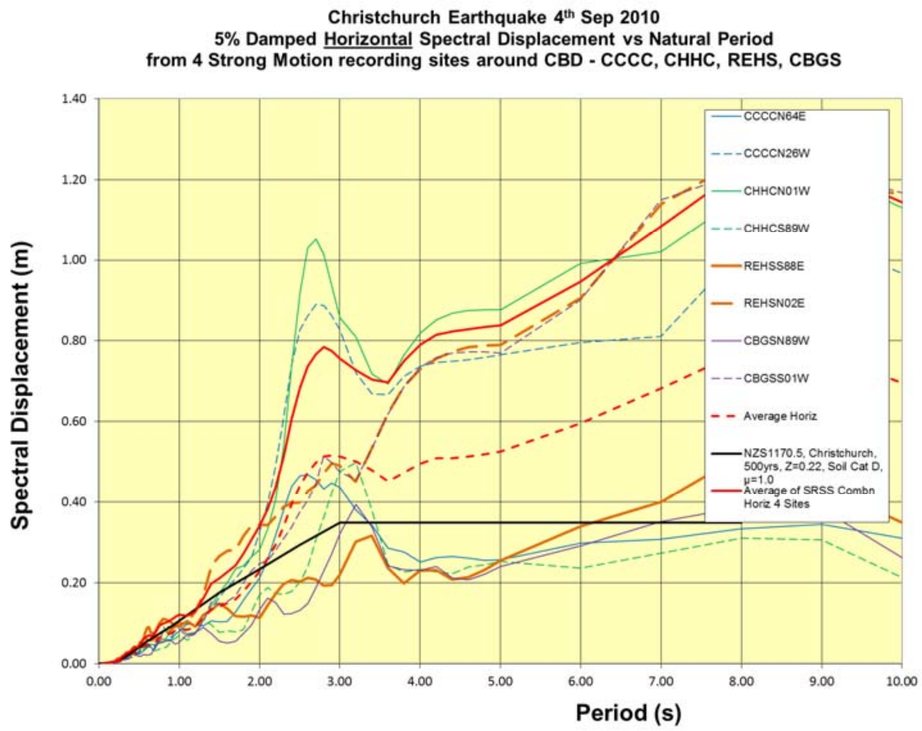


Figure 5.2 : Traditional Displacement Response Spectra for 4th September 2010 Earthquake at REHS Recording Site (5 % Damping)

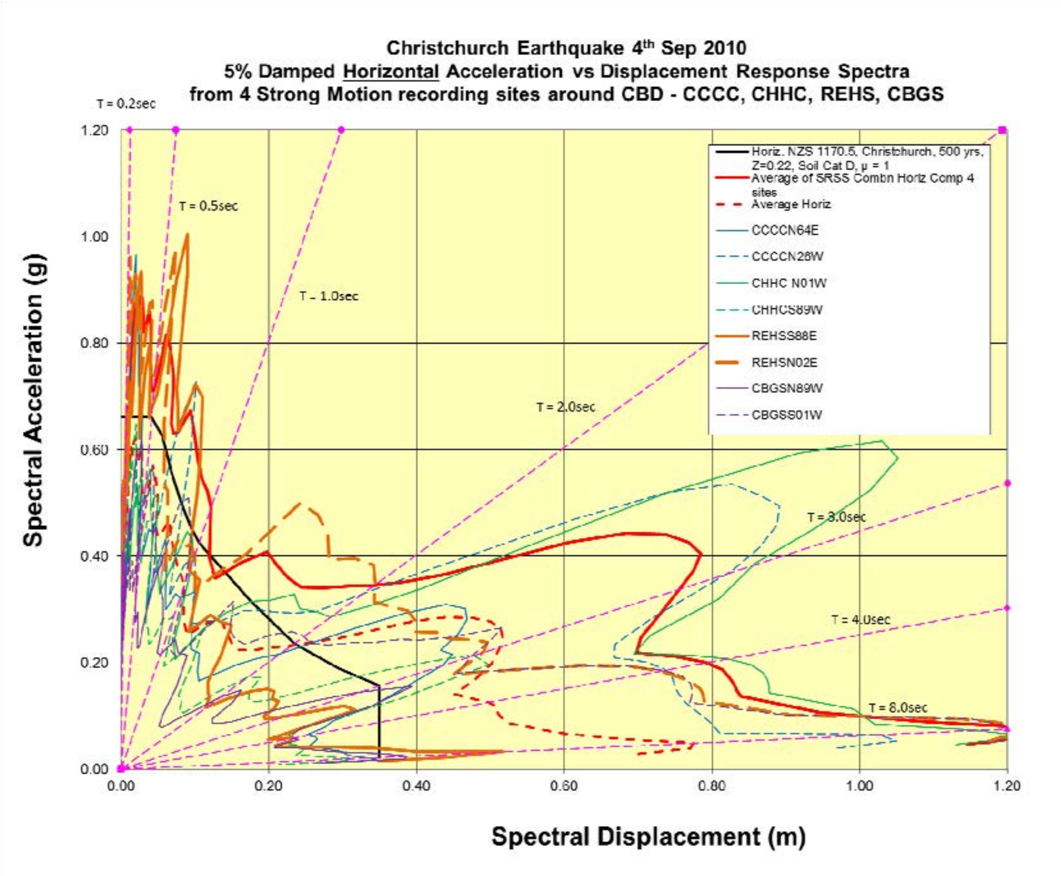


Figure 5.3 : Horizontal Acceleration-vs-Displacement Response Spectra from Recordings of the 4th September 2010 Earthquake (5 % Damping)

The straight lines radiating out from the origin in the bottom left corner of the figure each represent specific natural periods of the flexible structure. Radial lines for natural periods of 0.2, 0.5, 1, 2, 3, 4 and 8 seconds are shown (clockwise).

5.2 4th September 2010 and Aftershock Sequence

5.2.1 Earthquake Records

This Magnitude 7.1 earthquake with a focal depth of 10 km occurred at 4.35 am at a distance of 40 km from the building. An indication of the duration of strong shaking can be seen from GeoNet's plot from the REHS instrument which is nearest to the site (refer to Figure 5.4):

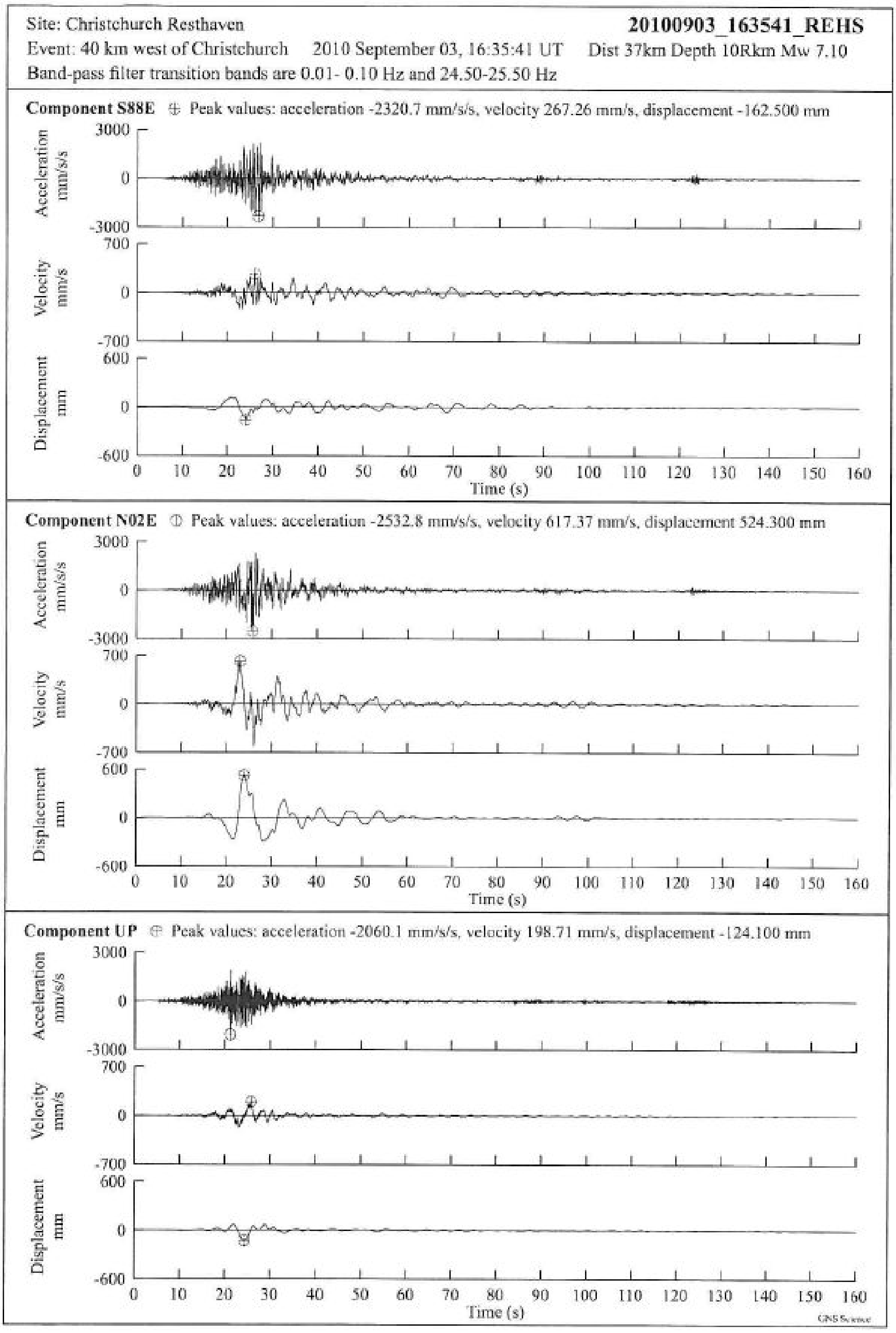


Figure 5.4 : Acceleration, Velocity and Displacement Records from the REHS Site

The horizontal response spectra for this earthquake have been shown earlier in this section. The acceleration-displacement response spectra for the vertical direction are shown in Figure 5.5 below:

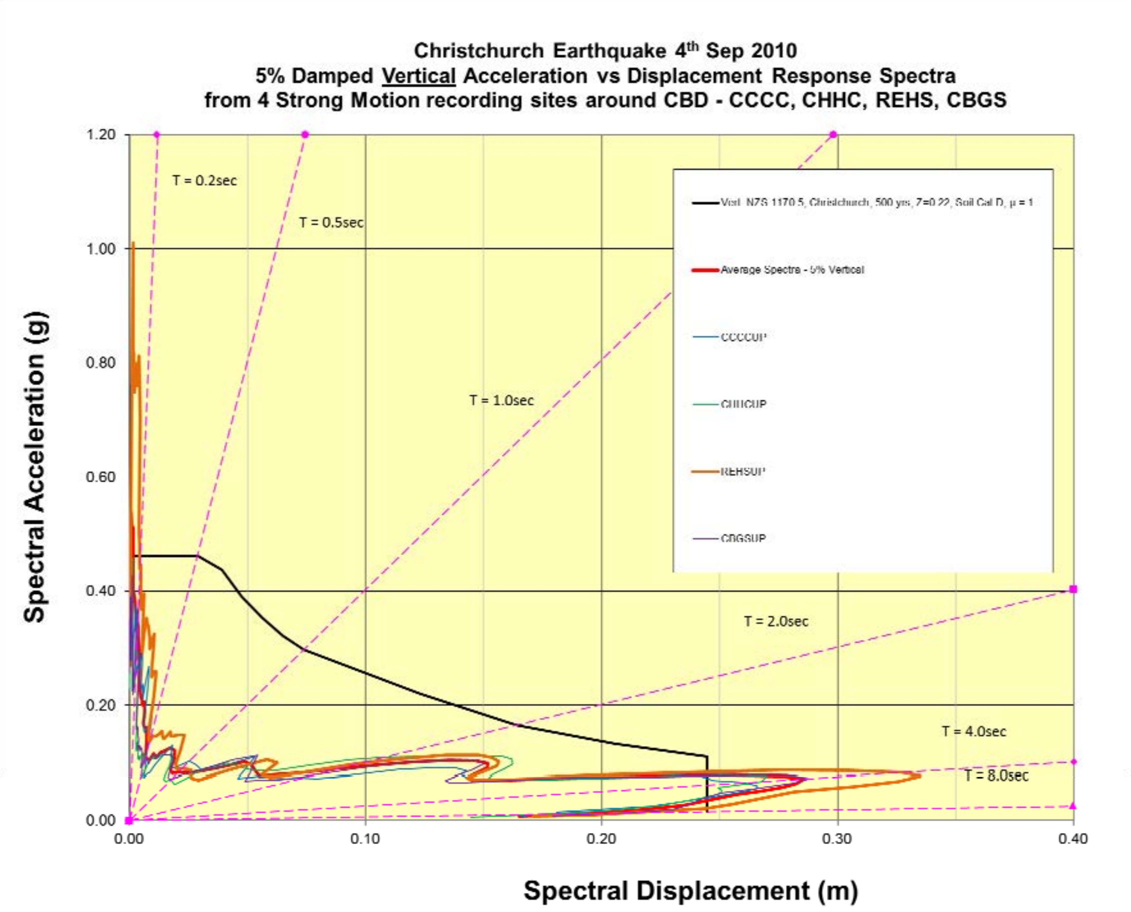


Figure 5.5 : Vertical Acceleration-vs-Displacement Response Spectra from Recordings of the 4th September 2010 Earthquake (5 % Damping)

5.2.2 Observed Building Performance

Engineers' Site Reports

A *Site Report* prepared by the building owner's engineer, dated 7th September 2010, records that a rapid structural assessment had been undertaken by a *walk around of the exterior, ground, first, fourth floors*, and that the inspection showed:

- *Cracks to ground floor and first floor level shear walls.*
- *Fourth floor ceiling grid bracing has failed, ceiling tiles have been removed, electrical and air conditioning systems are exposed.*

The report concluded: *Confirming 'green placard' building okay to occupy (structurally)*.

No Christchurch City Council Rapid Assessment forms corresponding to the original placarding (probably undertaken on 5th September) or the 7th September inspection have been sighted.

A Site Report of 16th September by the owner's engineer but by a different individual describes a *Re-visit to inspect new/growing cracks. RC frame around lift core.* It observes:

- *All cracks observed minor in shear walls – typically < 0.5 mm.*
- *One single crack 0.6 mm and minor spalling initiated at intersection approximately 100 x 100 x 10 mm max depth.*
- *Spalling in spandrel beams (outside) initiated by reinforcing corrosion – not significant.*
- *Inspected parapet above carparks on boundary. All cracks evident are old (moss in cracks) and wall is stable and has not moved.*
- *Okay to park below.*

The same engineer completed a *Christchurch Eq RAPID Assessment Form – LEVEL 2* on the same day. This summarised the above information, estimated the *Overall Building Damage (excluding contents)* to be in 0-1 % range, recorded the existing placard as Green (*Inspected*), and assigned a Green (G1) (*Inspected)* posting.

The owner's structural engineer's Site Report of 15th October by the engineer who undertook the 7th September site visit details a *Re-inspection of ground floor window gap and second floor partition crack*. It concludes that there were no structural issues, and that "*The building remains structurally okay to occupy on above observations*".

To assist with this (DBH) investigation, the owner's engineer has collated some photos taken on 16th September by its staff and, from recollection, annotated them to show the cracking and spalling referred to in their Site Reports. Figures 5.6 (a) and (b) are indicative of the damage shown in the owner's engineer's photographs. The position of the cracking shown in Figure 5.6 is not known with certainty.

Owner's engineer

Owner's engineer

(a) Cracking to Level One Shear Wall in Storeroom (inside the Southern End of the Shear-Core). Crack Widths between 0.2 and 0.6 mm.

(b) Cracking to Level One Shear Wall. Typically less than 0.2 mm.

Figure 5.6 : Indicative Damage Following 4th September Earthquake

Evidence from Public Witnesses

A number of members of the public contacted the Department of Building and Housing after the collapse with respect to concerns they had after the September earthquake. They included occupants of the adjacent Ernst & Young building, tenants of the PGC building, and unrelated observers.

The general theme was that they had noticed damage to the building after the 4th September earthquake. We have interacted with them by telephone and e-mail.

One of the respondents marked up a pre-collapse photo supplied by Beca with arrows showing where damage had been observed on the East face as seen from Manchester Street. The interfaces between the floor slabs and the top and the bottom of the external concrete columns at the upper levels were indicated.

Others identified non-structural damage to external window frames which was also reported by the owner's structural engineer's post-earthquake report.

5.3 Boxing Day, 26th December 2010

5.3.1 Earthqu ake Records

An indication of the duration of strong shaking can be seen from the GeoNet instrument CCCC which was closest to the epicentre of this Magnitude 4.9 earthquake. Refer to Figure 5.7.

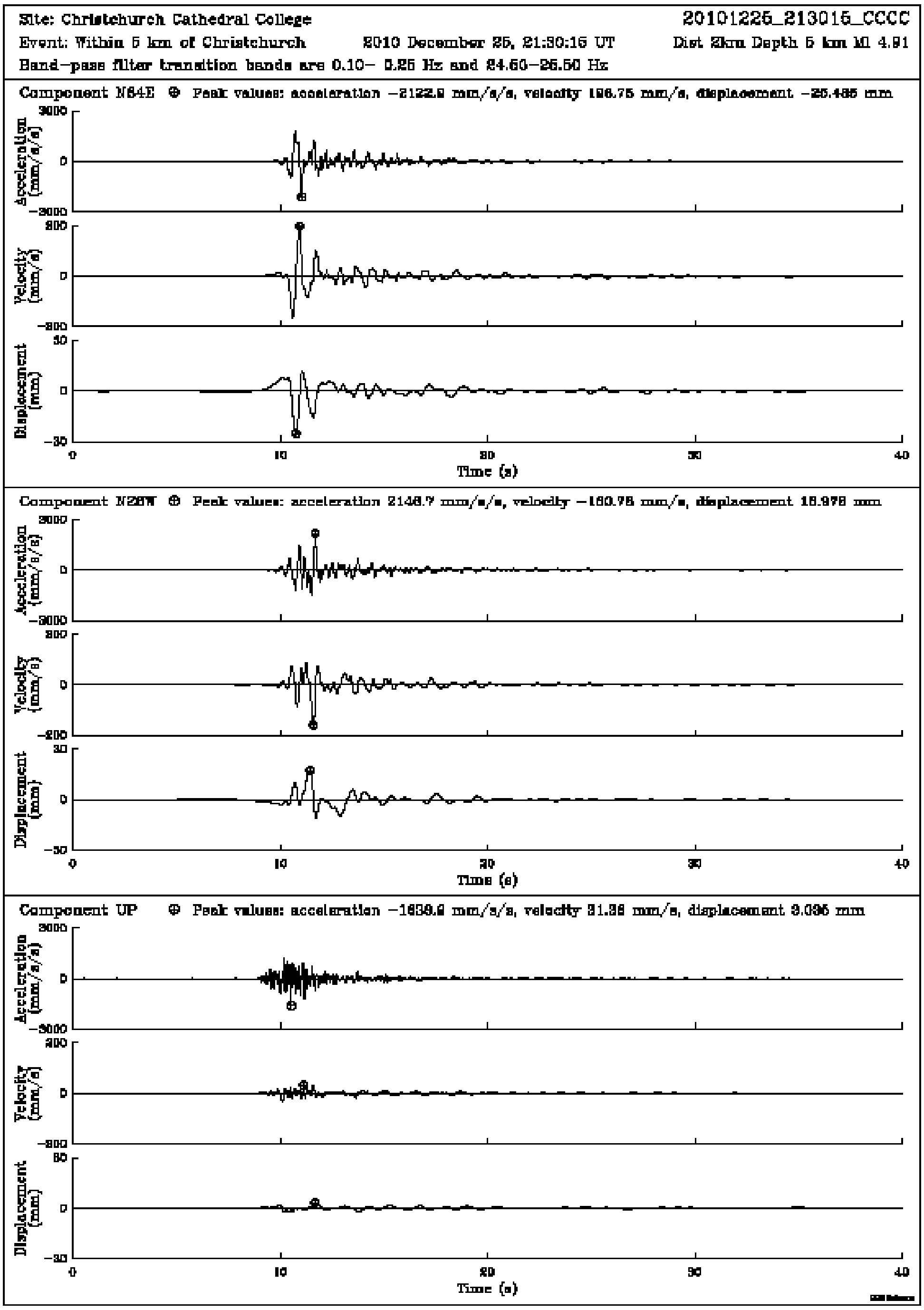


Figure 5.7 : Acceleration, Velocity and Displacement Records from the CCCC site

Figures 5.8 and 5.9 below show the Acceleration-vs-Displacement response spectra for this earthquake.

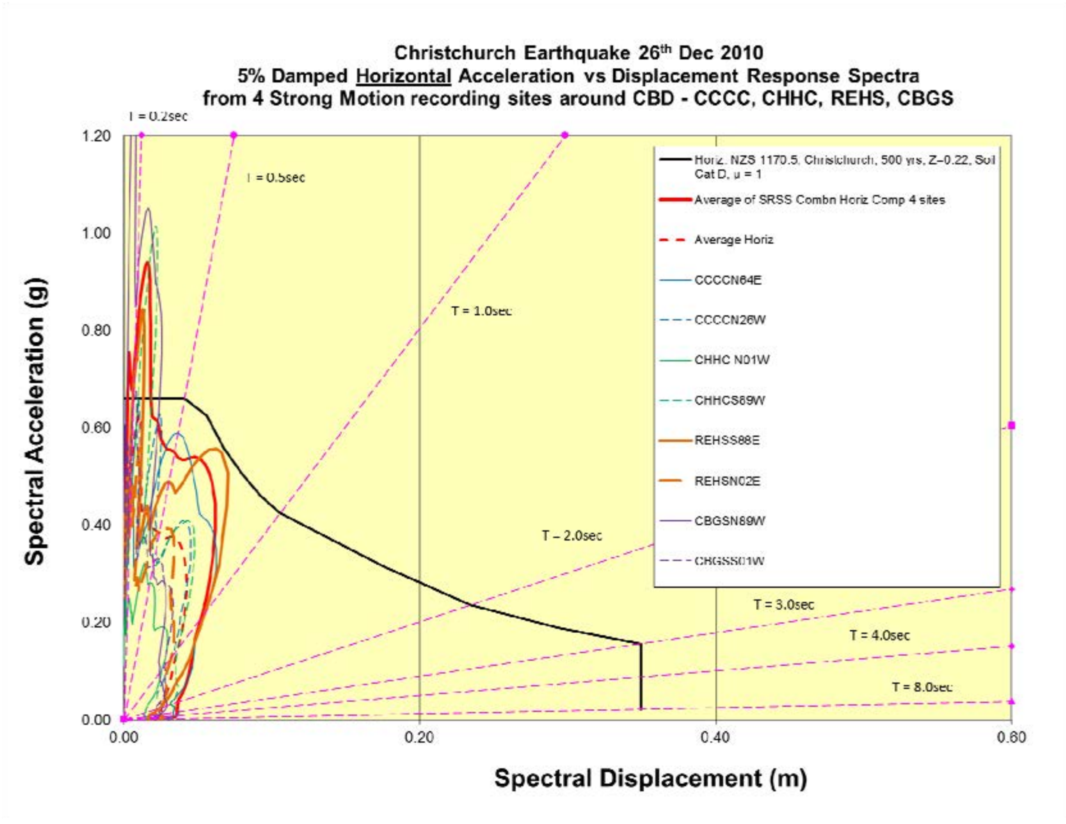


Figure 5.8 : Horizontal Acceleration-vs-Displacement Response Spectra from Recordings on 26th December 2010 (5 % damping)

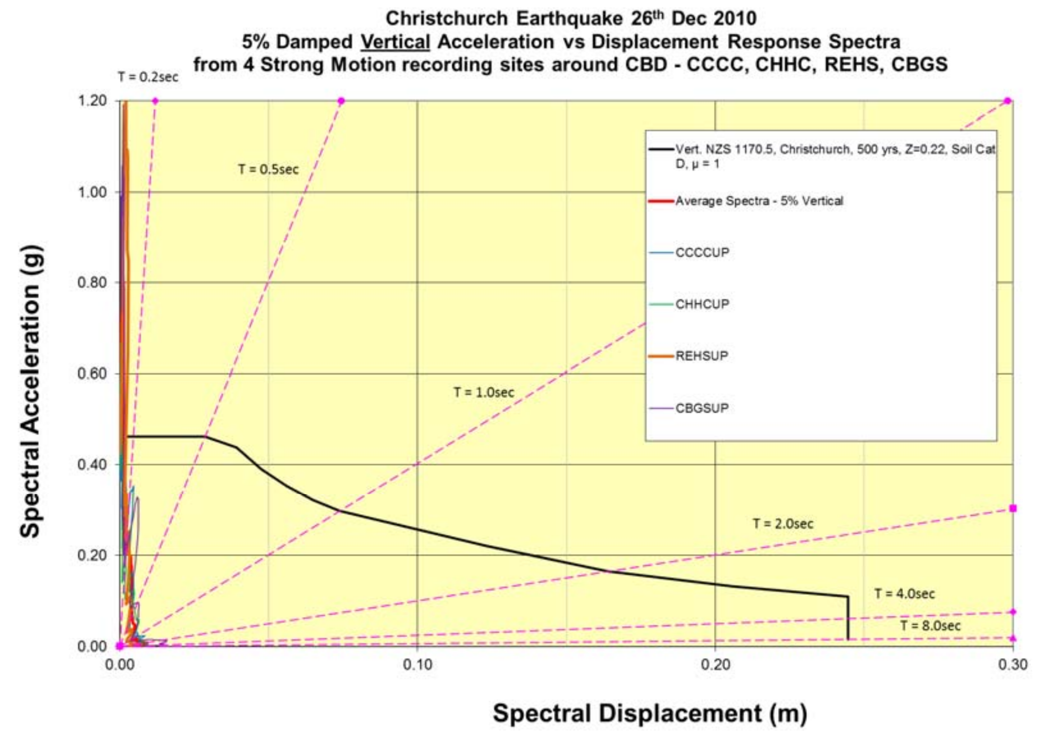


Figure 5.9 : Vertical Response Spectra from Recordings on 22nd February 2011 (5 % damping)

5.3.2 Observed Building Performance

Engineers' Site Reports

A Site Report by the owner's engineer dated 27th January 2011 (by the same individual as on 7th September and 15th October) details a *Re-inspection of previously observed damage level 1 and new cracks*. It reports:

- *Previous cracks have enlarged. Cracks to level 1 stationary (sic) wall now > .2 mm, minor spalling also evident. General diagonal cracking to all shear walls.*
- *New cracks to stair connection at level 1 – spalled plaster. Hairline cracks to most landings (stairs appear tied to all floors).*
- *Building remains safe to occupy.*
- *Cracks to shear walls greater than .2 mm will require epoxy injection repairs.*
- *Cracks to stairs should be repaired also where greater than .2 mm.*

Evidence from Public Witnesses

Occupants of the Ernst & Young building have advised that they noticed increasing damage (believed to be cracking of concrete) over the period from Boxing Day until the 22nd February 2011. The locations of the damage they observed have been identified as being at the bottom of some of the columns above Level One.

An occupant of the PGC building has stated that the building became more responsive (in a new way) to aftershocks in January and February than it had been before the September earthquake.

6 Effects of 22nd February 2011 Earthquake at PGC Site

6.1.1 Earthqu ake Records

This Magnitude 6.3 earthquake occurred at 12.51 pm, and its epicentre was approximately 10 kilometres south-east of the building site at a focal depth of five kilometres. An indication of the duration of strong shaking can be seen from the GeoNet instrument REHS about 670 metres to the north of the building site.

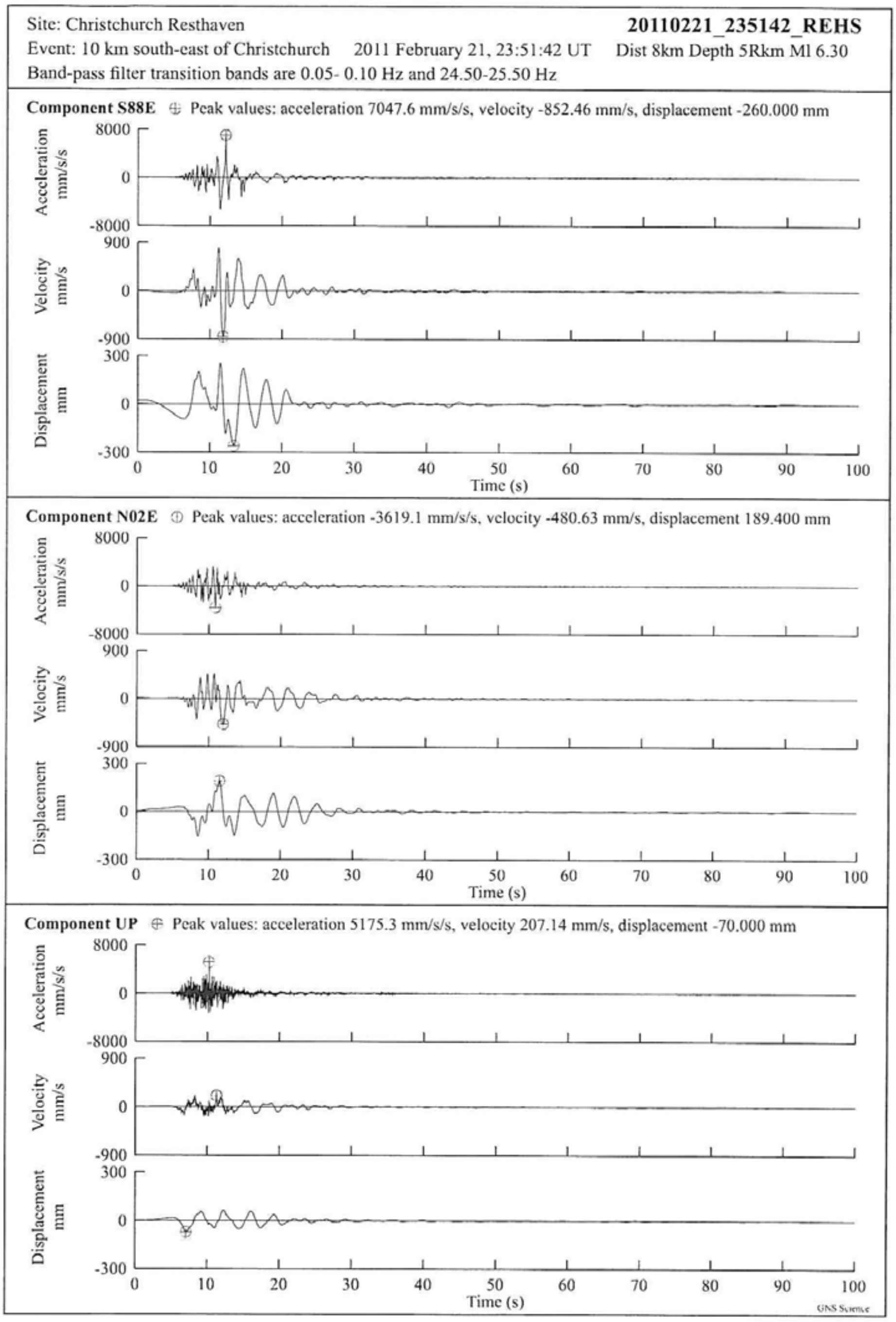


Figure 6.1 : Acceleration, Velocity and Displacement Records from the REHS site

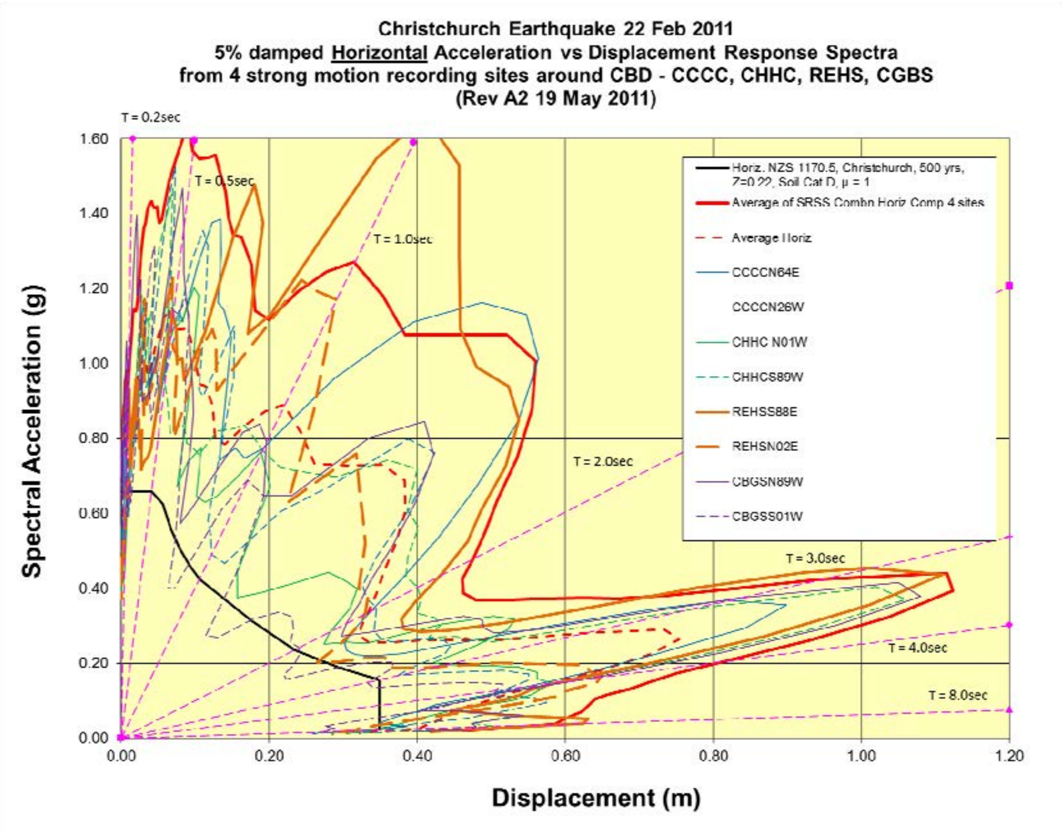


Figure 6.2 : Horizontal Acceleration-vs-Displacement Response Spectra from Recordings on 22nd February 2011 (5 % damping)

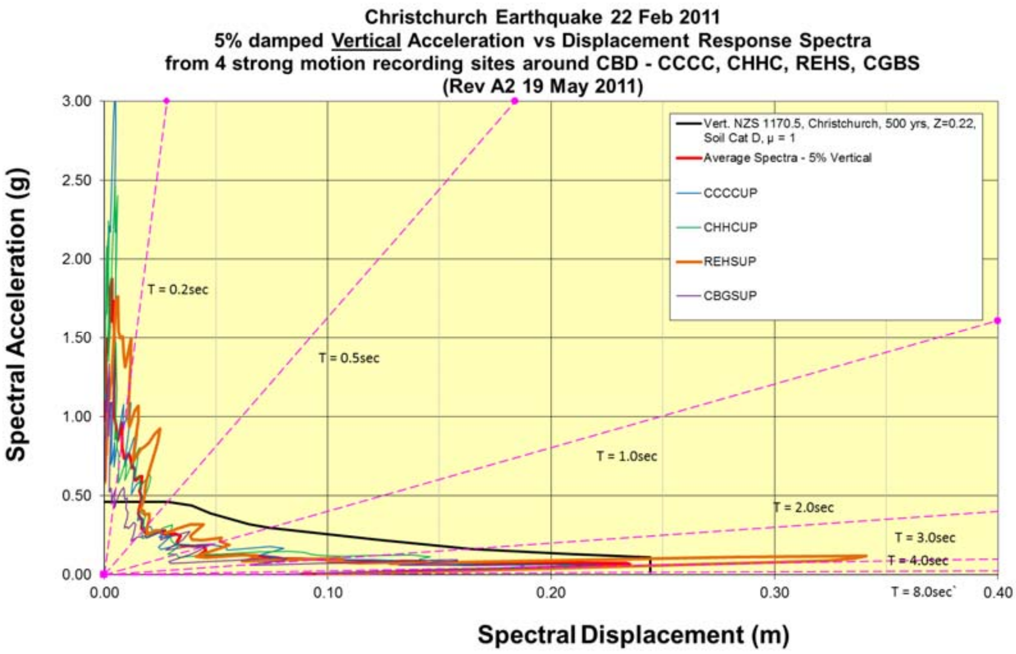


Figure 6.3 : Vertical Acceleration-vs-Displacement Response Spectra from Recordings on 22nd February 2011 (5 % damping)

6.1.2 Observed Site Performance

Evidence from Public Witnesses

Six observers of the actual collapse of the PGC building from directly across the Avon River were interviewed during April. Some of these were structural engineers. They were all asked to describe what they saw. It was made clear to them that we were particularly interested in the timing and mode/initiation of collapse.

There is no strong consistency between the observations. Some have described some torsional response, and others saw predominantly lateral (non-torsional) response. Most said that the collapse occurred during the middle of the strong shaking, but one thought that it had occurred after the most intense shaking they were experiencing had finished. One observer reported further collapse may have occurred in the first major aftershock ten or so minutes after the main shock. An observer (not interviewed) in the Ernst & Young building wrote that, after some lesser shaking, the PGC building was observed to sag in one motion away from the Ernst & Young building.

USAR Engineers Interviews

A meeting of New Zealand structural engineers embedded in the New Zealand Urban Search and Rescue (USAR) task forces was convened in April 2011 to discuss their observations of all four buildings being investigated by the Department of Building and Housing. Prior to the meeting, questions were submitted in writing to the engineers.

Two of the engineers were particularly involved with the PGC building, one from a few hours after the collapse, and the other from around 3.30 am the next morning. The following salient points about the collapse mode were made:

- The ground floor (to first floor) was structurally intact with no obvious deformation.
- The stair/lift core wall on the east side had failed only between the first and second floors.
- The walls above the second floor appeared to be dimensionally intact, and the stairs above the second floor were useable, although they were not thought to be safe.
- It was recalled that the doorway openings on the west side of the core retained their heights.
- The first-to-second floor east wall had not punched through the first floor slab – indicating that the collapse/failure was above the first floor, and within the wall itself.
- The roof on the east side of the core had detached and slid off the building onto the adjacent building.
- The connection between the floor slabs and the core had failed, and appeared to be not substantial.
- The building collapsed to the east with almost no evidence of rotation (about a vertical axis).

Investigators' Observations

From our own observations of the site in the days after the collapse from across the river, and from photos by others, the building's collapse eastwards appears to have been consistent with the failure of the eastern core wall between Levels One and Two. The eastern half of the roof detached itself from the core and slid partially off the level below onto the adjacent building. The predominantly gravity-only frame supporting the floors yielded at the joints between the columns and beams.

At the beginning of the investigation in April 2011, a visual inspection of the site, footpath and the roadway (Cambridge Terrace) to the south of the site was undertaken. Evidence of liquefaction or lateral spreading was not observed. Almost no structural damage was observed between Ground

Level, and Level One. At this time, demolition to Level One had been completed. On the east side of the core, demolition to Ground Level was complete, although not all cleared away.

7 Collapse Description

This is best described by reference to photographs taken immediately after the collapse, and before the demolition that preceded the start of this investigation.

The observations from Figures 7.1 and 7.2 are as follows:

- The shear-core had rotated about the west wall at Level One and was finally left at an angle of approximately 68 degrees to the horizontal.
- The shear-core east wall was lost between Level One and Level Two
- The floor slabs at roof and at Levels Four, Three, Two and One were displaced to the east by approximately 9, 4.5, 3.8, 2.5 and 0 metres, respectively.
- The floor slabs to the west were stacked almost on top of one another indicating very little rotation in plan occurred (i.e., collapse occurred almost entirely along the east-west axis of the building).

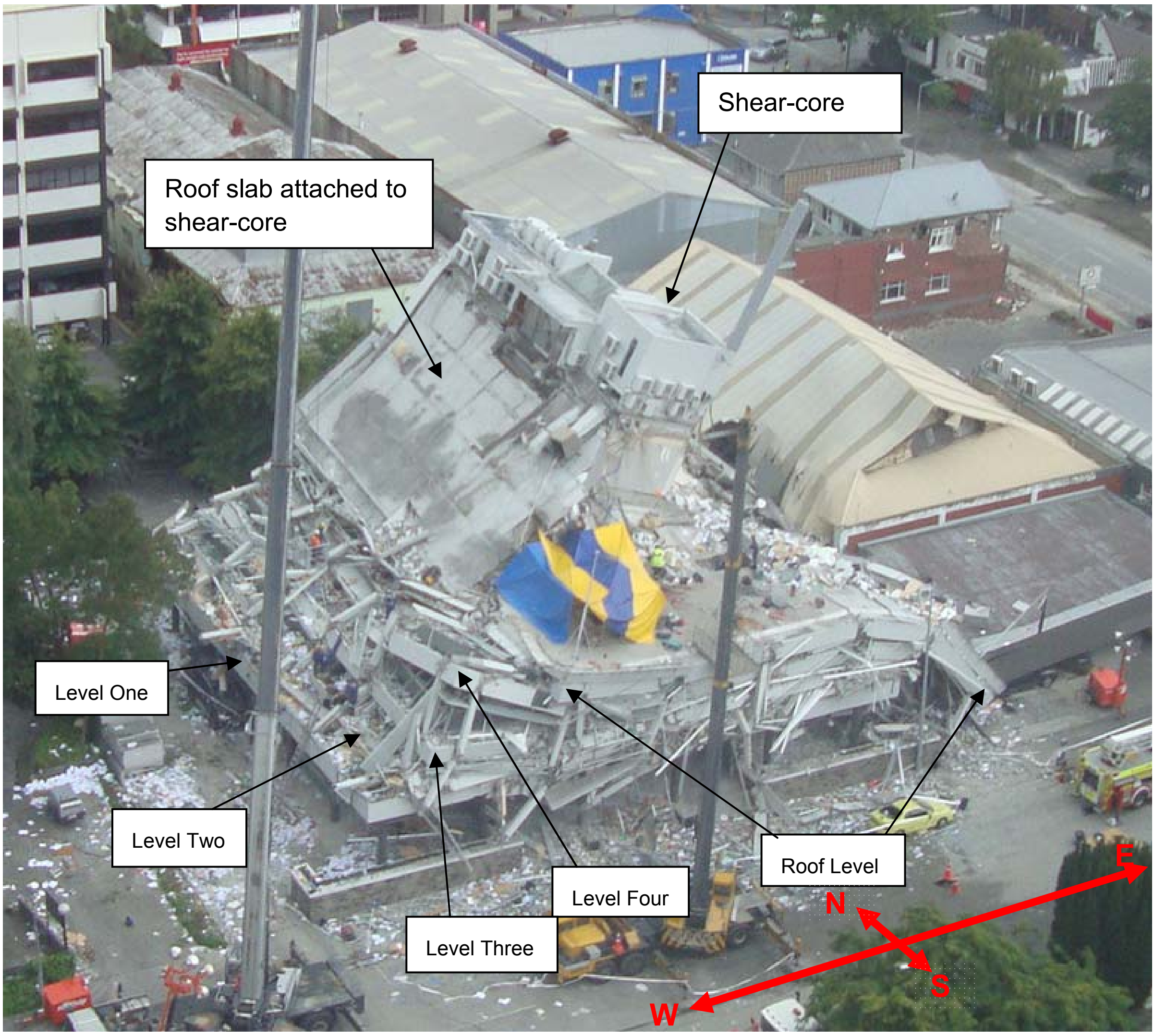


Figure 7.1 : General Location of Main Structural Components after Collapse

- The slabs detached from the east and south sides of the shear-core. The roof slab on the east side of the shear-core detached and slid over the collapsed floors below into the adjacent building.
- The roof slab remained attached to the west side of the shear-core. Photographs taken from the north indicate that the other slabs may have also remained attached on this side but this has not been confirmed.
- The joint regions between the perimeter columns and beams suffered extensive damage (spalling of concrete), with a number of columns punching into the joint or completely detaching from the beams.
- There was evidence of slab reinforcement, close to the slab surface soffit having been pulled from the concrete.
- There was also evidence of slab reinforcement having fractured at the junction between the slab and wall.

Eye witness accounts indicate that there was little disturbance in the structure below Level One and that the door openings from the core to the floors on the west side appeared to have retained their height on levels above Level One.

ex eye witness

(a) View from the South

ex USAR engineers

(b) View from North-West showing Floors still Attached to Shear-Core on West Side

Figure 7.2 : Various Views of the Collapsed Building

cont….

ex USAR engineers

(c) View from the North-East showing Levels Two, Three and Four Floor Slabs still Attached to Shear-Core. Note Loss of Shear-Core East Wall between Levels One and Two

ex USAR engineers

(d) View from South-West

ex USAR engineers

(e) View from South-East

ex USAR engineers

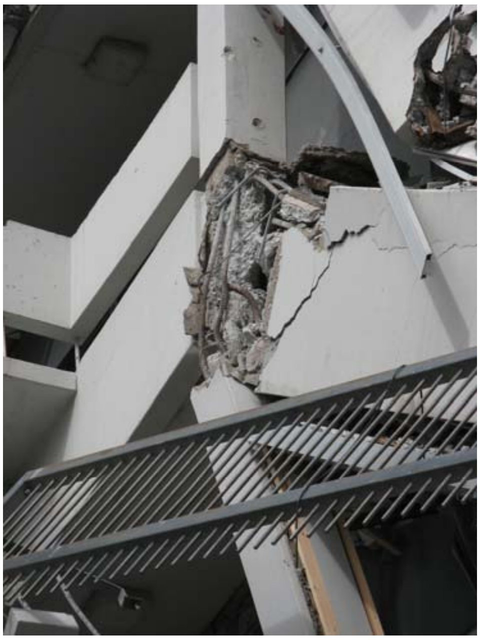
(f) View from South-West

Figure 7.2 : Various Views of the Collapsed Building

cont….

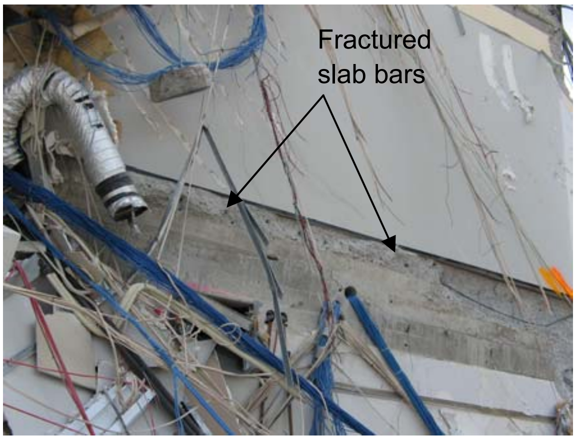
ex USAR engineers

(g) Soffit of Slab Showing where Slab Reinforcement has been Ripped from Slab



ex USAR engineers

(h) Typical Perimeter Beam/Column Joint. Note Lack of Joint Ties



ex USAR engineers

(i) Fractured Slab Reinforcement at Wall/Roof Slab Interface on the East Side of the Shear-Core

Figure 7.2 : Various Views of the Collapsed Building

8 Evaluation/Analysis

8.1 Structure Condition/Capacity prior to Collapse

8.1.1 Original Design

The building appears to have complied generally with the design standards and practices of 1963 which is the date of the structural drawings. A specific investigation of the adequacy of the original design of all the elements for vertical ("dead" and "live") loads has not been undertaken. However, the analyses undertaken for earthquake loading to determine the collapse mechanism on 22nd February 2011 confirm that the structure met the earthquake load requirements of 1963 and also of 1965 - if the requirements of the later Standard had been applied.

Strength tests post-collapse of concrete and reinforcing steel taken from specific structural elements (including the central walls) give values that are consistent with design assumptions that would have been common for a building of this type at the time of design. The test results are included in a separate report prepared by Hyland Consultants Ltd. Testing of shear-core reinforcement and concrete obtained from samples recovered from the landfill storage site was also carried out. These test results are reported in Appendix A1.2.

The position of the main structural elements in plan, including the eccentricity of the central walls and the cantilevered first level, were not unusual for the time – which was a few years before an emphasis on such matters for earthquake performance emerged.

There was information available (soil bores) to the structural engineer about the soils beneath the site, and no reason has been found to suggest that this was not used to determine the foundation design.

Nothing has been found to suggest that the construction was carried out other than in accordance with the plans.

8.1.2 1997 Structural Report

The building owner's engineer undertook a "Seismic Evaluation of Existing Building" in April 1997, and their report (Revision 0) has been made available.

The following is the Executive Summary from the report:

The Christchurch Drainage Board office building in Cambridge Terrace has been evaluated for earthquake effects based on the requirements of NZS4203. The evaluations have shown that:

1. *Column plastic rotations in the gravity columns exceed their capacity for earthquakes with a return period of from 35 to 70 years (one-third to one-half NZS4203 loading). The consequences of this are severe as the columns would lose gravity support capabilities leading to extensive collapse.*
2. *Wall shear cracking also initiates at relatively low loads. Cracking is generally limited to coupling beams and around openings. This cracking would lead to permanent damage but the consequences are not as severe as column damage as the wall portions support only small tributary areas of gravity load.*

An alternative gravity load support should be provided as a matter of some urgency given the small return period for severe damage and the consequences of this damage.

Strengthening the shear walls be (sic) adding concrete to the wall face will reduce damage to the walls but not eliminate it unless all walls are strengthened. Wall strengthening will not significantly reduce the danger of column collapse as foundation rocking and wall flexural yielding imposes rotations on columns regardless of wall shear strength.

The earthquake resilience reported in 1997 with respect to NZS4203:1992 would mean that the structure would now be classified as earthquake-prone (i.e., with earthquake capacity less than one-third of the 2010 new building standard). Our analyses reported on below confirm that this is a possibility, but suggest that the seismic resistance may have been slightly higher than predicted in 1997, and therefore designation above the earthquake-prone level may have been a possibility in 2010. This is primarily because our analyses do not predict failure of the columns until after the shear-core fails.

8.1.3 Subsequent Modification to Structure

After the 1997 structural review, the concern about the load capacity of the external columns was addressed by the installation of (18 per floor) steel posts behind a majority of the columns between Level One and the roof.

At the same time, some new openings were cut into the walls of the central core between the Ground Level and Level One, and others were filled in. The potentially most significant of these was the cutting of a new access opening through the west wall of the shear-core on Level One.

No evidence has been found to suggest that these changes materially reduced the inherent seismic resilience of the structure. Similarly, no evidence was found that these modifications contributed in a significant way to the cause of the collapse on 22nd February 2011.

8.1.4 Assessed Performance of the Building

The building, in its configuration immediately prior to the September earthquake, has been assessed against the current building standard, NZS1170.5 (as at 3rd September 2010), in accordance with the requirements of the NZSEE guidelines (refer Appendix A5). This assessment indicates that the building can be shown to have had a capacity in the range 30 to 50*%NBS* (New Building Standard) depending on the level of performance compared. At the higher end of the range (i.e., at the point at which all of the tension steel in the shear-core has theoretically fractured) there is little resilience available to sustain any increase in shaking intensity and therefore the building capacity might be more appropriately described as being in the tighter range of 30 to 40*%NBS*.

8.2 Effects of Earthquake on Site and Structure

We have carried out a number of structural analyses of this building. These are reported in Appendix A4.

All analyses undertaken show that the shaking experienced by the Pyne Gould Corporation building structure on 22nd February 2011 was well in excess of that for which it had been designed.

Whether or not a building actually collapses in an earthquake that generally exceeds its design resilience depends greatly on the unique characteristics of the shaking at the particular site. These characteristics include:

- Duration and number of cycles of strong shaking.
- Predominant frequencies of ground shaking.
- Any directionality in shaking.

The extent and location of the consequent damage also determines whether the building becomes just severely damaged, or whether it catastrophically collapses.

The analyses undertaken of the 4th September 2010 and 26th December 2010 (Boxing Day) earthquakes show that the building may have suffered some minor to moderate damage (consistent with the photographs taken by the owner's engineer following the September earthquake), but this would not have been sufficient to cause or contribute to the later collapse.

The same analyses suggest that the building would not have collapsed if the shaking experienced in these earthquakes had not been greater than that consistent with the 1963 design levels.

Analysis for a level of shaking consistent with the 2010 new building design levels (as specified in NZS 1170.5:2004) has not been undertaken.

The owner's structural engineers provided reports on the inspection of the building after both the 4th September 2010 and the Boxing Day earthquakes. Positions of cracks observed were noted and some photographs were taken (refer to Figure 5.6).

While there was some cracking of the shear-core walls documented after the 4th September 2010 earthquake (sufficient to possibly indicate some bar yield but not bar fracture), and probably relatively small cracking in some columns, it cannot be positively adduced that this damage, even if it was assumed that it was in the most unfavourable location, weakened the structure significantly to the extent that it caused or contributed to the collapse on the 22nd February 2011.

Similarly, the Boxing Day earthquake does not appear to have weakened the structure to the extent that it would have led to a different outcome on the 22nd February 2011.

From the information available, it is considered unlikely that the level of damage that had occurred would have provided any warning of the collapse that was to occur.

The analyses completed as part of this investigation indicate that the flexible columns above Level One are protected by the stiffer shear-core until the point at which the shear-core rotates at Level One. Refer to Appendix A5 for further discussion on the capacity of these columns.

The geotechnical investigations have shown that good ground conditions exist on this site, with no evidence that the building had settled over its life or that liquefaction had occurred under the building during the earthquakes. These investigations also included inspection of the interface between the soffit of the ground floor slab and the supporting soils, and could not identify any sign of relative movement.

The vertical accelerations were high during both the September and February earthquakes. However, the time-history analyses indicate that the level of vertical acceleration experienced by this building was unlikely to be a contributory factor in its collapse. A possible reason for this is the non-coincidence of the peak vertical and horizontal accelerations.

9 Consultant Evaluation of Collapse and Possible Reasons for it

From the photographs of the building in its collapsed state, comments by eye witnesses and the results of the various analyses that have been completed, the following collapse sequence is inferred. Reference to Figure 9.1 is made in the following discussion. The times noted have come from the inelastic time-history analyses, and should be considered indicative only.

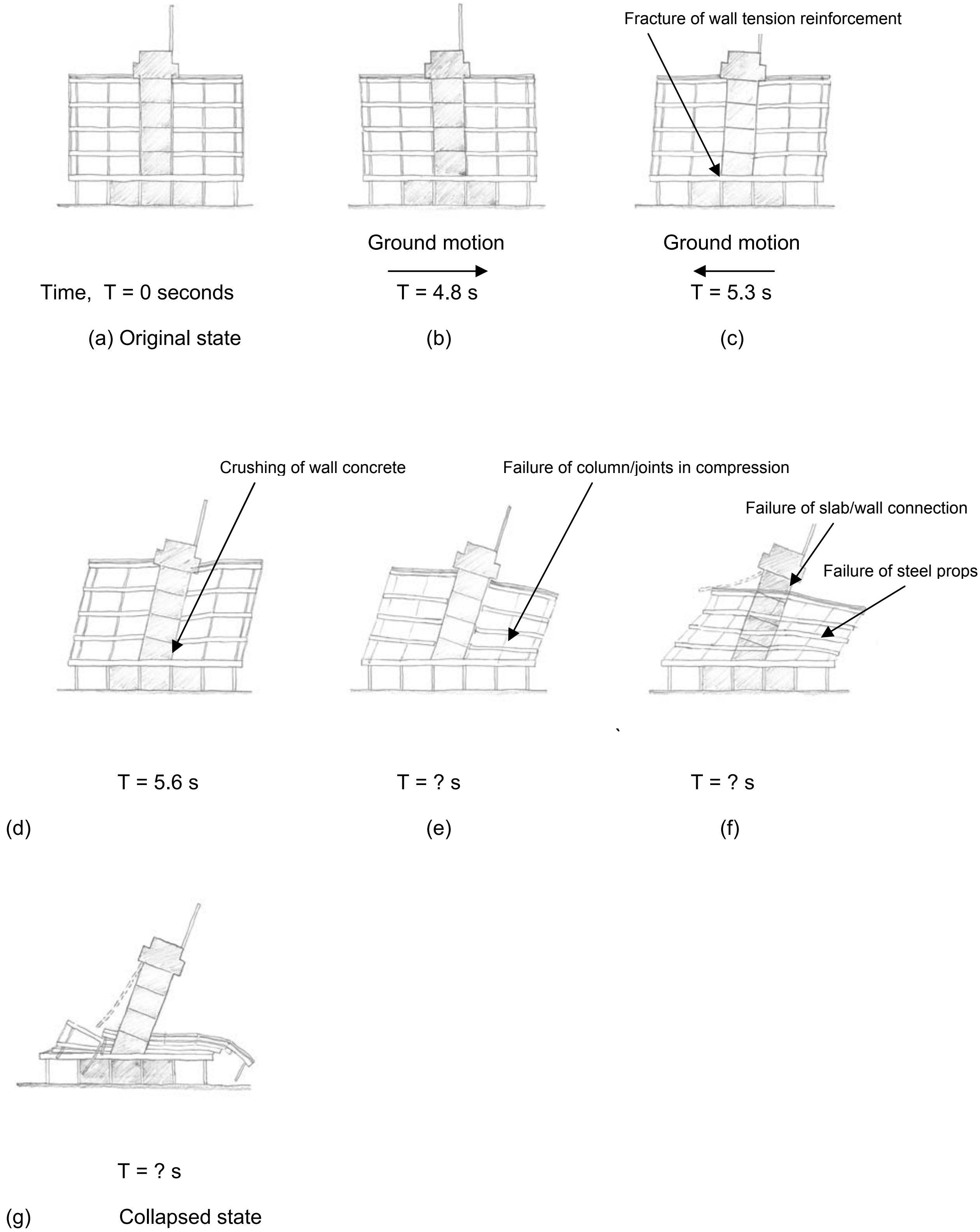


Figure 9.1 : Inferred Collapse Sequence

- In the early stages, the building experienced violent shaking that caused various parts of the shear-core to yield (i.e., to stretch plastically). This is likely to have resulted in moderate cracking, but this has not been able to be confirmed.
- Approximately five seconds after shaking commenced, the building experienced a large horizontal ground displacement pulse to the east (sufficient to crack the concrete immediately above Level One on the east side of the shear-core), followed almost immediately by ground movement to the west. Refer to Figures 9.1(b) and 9.1(c).
- This caused the vertical reinforcing bars in the west wall of the shear-core immediately above Level One to yield and fracture. Refer to Figure 9.1(c).
- The shear-core continued to displace to the east, rocking about the east wall of the shear-core immediately above Level One until the compression capacity of the concrete in this region was exceeded and failed. Refer to Figure 9.1(d).
- Simultaneously the base regions of the columns at Level Two yielded. Refer to Figure 9.1(e).
- The shear-core continued to displace to the east because the building was totally reliant on the shear-core for its lateral resistance. The frame had insufficient strength or stiffness to contribute - so resistance to overturning was completely lost.
- The perimeter columns had insufficient strength to yield the beams and insufficient ductile capability to sustain the imposed distortions and would no longer have been able to carry the imposed vertical gravity loads. Likewise, the joints had no binding steel and therefore the outer column reinforcing bars had no restraint against buckling once they passed into the joint and the cover concrete spalled.
- The floor slabs and frame were forced to follow the displacing shear-core until either vertical support for the Level Two columns (and steel props) was lost, or the connection of the floor slabs to the shear-core failed or both. Refer to Figure 9.1(f).
- The steel props continued to hold some of the floors apart but, having no rotational resistance at each end, could not resist the movement of the floors to the east, and just rotated as the floors moved further to the east. Refer to Figure 9.1(f).
- The roof floor slab on the east side, having detached from the shear-core, slid down and over the collapsed lower floors and onto the adjacent building.
- The roof slab on the west side of the structure was still attached to the shear-core, and was forced to follow the shear-core as it rotated over until the displacements were such that the columns and steel props could no longer provide vertical support.
- The building was now is a complete state of collapse. Refer to Figure 9.1(g).
- There was little or no damage to the structure between the Ground and Level One

This postulated sequence of collapse is supported by an eye-witness (positioned on the west side of the building on the top floor of Ernst and Young building when the earthquake happened) who reported that *all I saw was the top of the PCG building tilt slightly away from us and descend out of sight in what seemed a rather smooth and graceful motion*. Rescue personnel also reported that their recollection was that doorways on the west side of the core appeared to have retained their height (consistent with the wall failing by crushing) and that the ground floor of the building was almost undamaged.

The collapse sequence is also supported by the analyses that show that, under the strong motion record considered representative for the 22nd February event, the reinforcing steel in the west wall of the core could fracture and be followed by a compression failure of the wall concrete.

This building was particularly vulnerable for a number of reasons once the strength of the shear-core was exceeded. When these vulnerabilities are taken together, they could explain why this building failed when others with similar characteristics did not. The reasons are:

- The strength of the shear-core was low when compared with both current (2011) Code requirements and the demands of the 22nd February earthquake.
- The building was totally reliant on the shear-core for its horizontal resistance to earthquake.
- The concrete in the shear-core in the critical region was not confined by supplementary reinforcing as would be provided today.
- The level of vertical reinforcement provided did not add significantly to the overturning resistance of the shear-core.
- The thickness of the shear-core walls did not meet current recommendations to prevent instability (buckling) of the walls under very high compression stresses.
- The frame was sufficient only to prop the floors, and could not contribute significant resistance to horizontal earthquake shaking.
- The frame had almost no capability to deform without failing once the strengths of the columns and joints (i.e., the region at the junction between the floor beams and the columns) were exceeded.
- The presence of the steel props may have caused the floors to displace horizontally further than they might otherwise have done (thus delaying the floor collapse a little).
- The attachment of the slabs to the shear-core walls was insufficient to restrain the floors once the shear-core failed. There is evidence to suggest that, because the bottom slab bars were located very close to the soffit (underside) of the slab, they peeled off the bottom of the floor slab. If this also occurred for the top slab steel, it could have led to premature failure of the connection between the shear-core and the slab in tension and/or shear. There are photographs taken after the collapse which show fractured slab reinforcing bars at the shear-core wall face. Calculations confirm that bar fracture is possible at the wall/slab interface once the shear-core rotates relative to the slab.

While unusually high vertical accelerations were recorded at a number of places in Christchurch during this 22nd February earthquake, our computer analyses indicated that the presence or otherwise of the vertical accelerations does not influence the failure they predict for this earthquake.

10 Conclusions

10.1 Reasons for Collapse

The damage and crack widths described in the engineers' site reports, in our opinion, do not themselves indicate that the building suffered damage in the 4th September and Boxing Day earthquakes that would cause concern for a structural engineer considering whether the building was suitable for reoccupation.

The engineers' site reports do not identify any structural damage at the tops and bottoms of the perimeter reinforced concrete columns. Although one witness has reported such damage on the east side sufficient for it to be seen from a considerable distance, no other reports of this have been received. Photographs that might have corroborated such damage were not available.

The balance of probability is that the damage observed by the public before the 22nd February 2011 was due to relative (inter-storey) horizontal motion between floors – possibly from a small torsional response of the building which would have been greatest at the perimeter. The perimeter concrete columns were detailed to be no more than props (in current design terms), and would have exhibited some cracking without significant degradation of their propping capability (which had been previously found to be small enough to justify additional steel props being installed).

It is possible that the damage/cracking that occurred in the 4th September and Boxing Day earthquakes could have made the building perceptibly more responsive in the larger aftershocks experienced by occupants up to the 22nd February 2011. However, the cracks in the shear-core (after September) are unlikely to have led to an appreciable loss in horizontal stiffness, as the severity of the aftershock shaking was unlikely to have been sufficient to reopen the cracks.

The description of the mode of collapse by the occupant of the Ernst & Young building seems to be the most representative of the range of observations – that the PGC building experienced some very visible shaking during the most intense part of the 22nd February earthquake, and then steadily sagged towards the east to its final collapse position. There has not yet been corroboration of the observation that there was further collapse of the building after some minutes. In all probability, the core walls did not move further after the initial collapse. We cannot conclude from the comments made whether or not there was a substantial torsional response before the collapse.

In our opinion, the collapse was primarily due to four factors:

1. The shaking experienced on 22nd February was several times larger than the loads the building was designed to resist.
2. A compression or buckling failure in the east wall of the unconfined shear-core immediately above Level One.
3. The inability of the columns and joints in the perimeter frame to sustain the resulting horizontal displacements.
4. The inability of the slab-to-wall connection to sustain the imposed rotations, shears and tensions resulting from the forced displacement of the shear-core.

The possible fracture of the tension reinforcement in the shear-core is not likely to have been a significant factor and may have delayed the collapse as it would have allowed the wall to rock.

We have concluded that the perimeter frame was able to sustain the imposed building deformations up to the point that the compression failure occurred in the shear-core at Level One.

10.2 Pre-Warning of Collapse

We have reviewed the provided information on the damage sustained in the 4th September and Boxing Day earthquakes and have concluded that there were few, if any, signs that the building had been significantly distressed in the shaking that had occurred, or that collapse was a possibility.

10.3 Other Factors

We have concluded that the following factors were not significant contributors to the collapse:

- Ground conditions.
- Previous damage.
- Vertical accelerations.
- Modifications to the building structure, including the additional opening into the shear-core on Level One made in 1998.

11 Recommendations

11.1 Recommendations to DBH in Relation to Building Investigation, Design, Construction or Approvals

The benefits of an active approach to the screening of existing buildings for critical structural weaknesses (CSWs) have been highlighted. Territorial Authorities should be encouraged to include such an approach in their earthquake-prone building policies.

This building, which was designed and constructed in the 1960s, appeared to have been designed to the Standards of the day and be well-constructed. Nevertheless, it contained details that meant that it was particularly vulnerable.

It has underlined the importance of identifying and addressing buildings of this type that are likely to behave in a brittle fashion, and therefore will have little resilience once the capacity of the structure is exceeded.

It is recommended that existing building assessment guidelines be reviewed to confirm that buildings of the Pyne Gould Corporation Building type (i.e., lightly, centrally reinforced, shear-walls where horizontal seismic resistance is provided solely by the shear-walls) will be identified as potentially poorly-performing in earthquakes and, if necessary, the guidelines should be revised to ensure that this is achieved.

11.2 Other Recommendations

The performance of this building during the 22nd February earthquake has highlighted the potential vulnerability in large earthquakes of lightly, centrally reinforced, shear-walls without concrete confinement, especially where the horizontal resistance to earthquake is provided solely by the shear-wall.

Further investigation of the seismic performance of existing, lightly reinforced, shear-walls is considered a priority.

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