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Report

Investigation into the Collapse of the Forsyth Barr Building Stairs 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 stairs in the Forsyth Barr Building on the south-east corner of Armagh and Colombo Streets 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 an expert panel appointed by DBH to oversee investigations into four buildings severely damaged or collapsed in this earthquake.

The eighteen-storey building, designed in 1988, is founded on a shallow raft, and its lateral resilience is provided by the frame action of the reinforced concrete beams and columns. For three levels above the ground level, the floors extend beyond the footprint of the tower to form a podium on the south and east sides. The "scissor" stairs are orientated diagonally within the tower in a north-east/south-west direction. The majority of the stairs comprised precast units cast into the landing at their upper end, and seated on a steel channel at their lower end with a horizontal gap specified to be 30 mm wide.

The scissor stair system was the sole means of egress (other than the lifts) from the building

A building designed in 1988 could be expected to perform to essentially the same level as a similar building in 2010.

No evidence of significant structural changes being made to the building since its construction has been identified.

Minor structural damage was observed after the 4th September 2010 earthquake, including some cracking and deformation of a few flights of stairs. A web cleat weld failed on a beam supporting the vehicle ramp in the podium. The Level 1 rapid assessment undertaken within a few days of the earthquake under the authority of Civil Defence resulted in the building being tagged Red – Unsafe. This was changed to Yellow – Restricted Access in the course of completing the Level 2 rapid assessment undertaken by the property manager's structural engineer. After the structural engineer's recommended propping of the steel beam was completed and further inspection of the stairs completed, the building was re-tagged Green – Inspected.

Subsequently, the owner's structural engineer undertook an inspection of the building, and prepared instructions for the repair of cracked structural elements.

Building occupants interviewed have stated that repairs to earthquake damage to floor coverings on the stairs in the period between the September 2010 and February 2011 earthquakes were underway.

No reports of structural damage after the Boxing Day 2010 earthquake have been received.

In the 22nd February earthquake, both flights of stairs collapsed; one side of the stair well up to Level 13 and the other up to Level 15.

An Urban Search and Rescue (USAR) team removed the collapsed stairs some weeks later after breaking them in half at the intermediate landing. The removed stairs, then located in the car-park to the east of the building have been visually examined. Evidence of cutting/grinding of the lower ends of at least two units (presumably to increase the in-place seismic gap) has been seen.

Analytical models of the structure have been created by Beca, and have been subjected to the simulated effects of the two earthquakes; 4 Sept 2010 and 22 Feb 2011. In addition, analyses of a

typical stair unit have been undertaken to determine a) the effect of vertical accelerations, and b) to understand its failure mode if it were to be placed under compression and/or sideways bending.

Soils data was obtained to inform the structural analyses.

Because of the absence of stairs and the danger from the remaining stairs, access to the building was not easily available. In September 2011, we were able to inspect the stair seismic gaps at Levels 14, 15 and 16. Indications that the seismic gaps may have not been constructed in accordance with the drawings was noted. It was not possible, on this occasion, to determine the extent of damage to the building structure. We have received copies of reports prepared by the building owner's engineers that indicate the building structure is relatively undamaged.

In our opinion:

Original Design

- The stairs as designed met the 1988 design requirements for the prescribed earthquake loads.
- The precast stair units in the tower were cast into the floor at their upper levels, and free to slide horizontally, within limits, at their lower ends.
- 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

- We have viewed the stairs removed from the building after the 22nd February 2011 earthquake. We have inspected the seismic gaps at the lower landings of stair units still in place at Levels 14, 15 and 16.
- It would appear that the stair units were precast as one unit rather than two flights interconnected with a cast in-situ concrete mid-height landing. It is considered unlikely that this change had any effect on the collapse.
- There is evidence of modification to the lower end of at least four stair units that may indicate the prescribed seismic gap at that end was not achieved in all cases during construction.
- Repair of the floor coverings in the seismic gap areas of the landings was underway at the time of the 22nd February 2011 earthquake. No evidence that these repairs had an impact on the stair collapse has been identified during this investigation.

Comparison with Current Code

- The clearance requirements for stairs from structure are essentially the same as was the case in 1988.
- The seismic gap provided in the original design would not meet current requirements by a factor of approximately 1.2.

Damage prior to 22nd February 2011

- Damage to the stairs and the structure was observed and/or reported after the 4th September 2010 earthquake as follows:
  - Cracking and vertical displacement in some of the stair units and to the floor coverings at the landings.
  - Cracking in the main structural frame members.
  - Failure of a weld in the region of a carpark ramp.
- Inspections of the most-damaged stair units carried out immediately after the September earthquake did not indicate that there had been any significant movement at the lower support.

Damage after the 22nd February 2011 Earthquake

- The main stairs from the ground to Level 15 (on one side) and ground to Level 14 (on the other side) collapsed, bringing with them the light-weight wall between them in the stairwell.
- The upper part of a column supporting the south-east corner of the podium roof was significantly damaged.
- We have not inspected the interior of the building other than at Levels 14, 15 and 16, but we have sighted two reports dated 31 March 2011 and 13 April 2011 that have been prepared by the owner's engineer that describe the extent of damage to the structure.
- Our interpretation of these reports is that the damage to the structure is relatively minor.
- Laser scanning of the north and west façades of the building does not indicate any significant permanent distortion of the structure.
- The removal of the collapsed stair units necessitated cutting them in half at their middle landings, and no records are available of which units were already broken/damaged at their mid-height landings or from which levels the various pieces originated.

Mode of Collapse

- The sequence of the stairs collapsing has not been determined. It seems likely that the uppermost stair units collapsed first, possibly progressively spearing the units below.
- Interviews with occupants suggest that all the stair collapses occurred during the main shock over a short period of time.
- It is likely that support at the bottom landing of one or more units was lost first, allowing the unit to pivot downwards about its upper end which was cast into the upper landing. In most cases, the cast-in reinforcing steel at the upper landing has yielded and then snapped, presumably allowing the stair unit to fall down building in a near vertical attitude. We have been advised that at least some of the units did not detach from their upper connections and were left hanging in the stairwell until removed by USAR.
- On any one unit, the lower seating support could have been lost for one of (or combination of) three reasons:
  - A stair flight has been compressed, resulting in bending downwards and yielding of the reinforcement, because the seismic gap was smaller than needed in the earthquake of 22nd February 2011. The resultant permanent shortening of the flight was sufficient for the lower landing to fall off the steel seat on the reversal of the relative motion. Analyses completed by Beca indicate that inter-storey displacements (drifts) were likely to be highest between Levels 10 and 14.
  - The lower stair landing failed in shear when the unit was subjected to compression after the seismic gap was closed.
  - The effective horizontal length of the flight was shortened when struck by the flight above after the flight above lost its seating and rotated downwards about its upper landing. The consequent V-shaped lower flight would drag its lower landing off its seat.
  - A free-falling stair unit simply "pole-axed" the still-intact flight, causing it to fail catastrophically and fall.

Reasons for Collapse

- The damage observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes is not considered to have significantly weakened the stairs to make them more vulnerable in the 22nd February 2011 earthquake.
- The actual seismic gaps at the bottom landings were too small for the earthquake shaking experienced on 22nd February 2011.

- The stair units were not designed to resist compression that would arise from the closing up of the seismic gap.
- The characteristics of the lower seat did not allow any latitude if the building inter-storey displacements in an extreme event were such that they exceeded the gap provided.
- Construction tolerances and the possibility that the seismic gap at the lower stair support had been filled (construction debris or mortar), would have reduced the level of building horizontal displacement required to fail the stair.
- Our analyses predict that the stairs would have collapsed even if the gaps were clear of obstructions.

Commentary

- The seismic gap specified on the drawings met the prevailing design standards at the time the building was designed.
- We have been unable to definitively establish whether the specified gap was provided everywhere, and whether there was construction rubble/dirt/mortar in the gaps that would have reduced their effectiveness.
- The specified gap would have not been sufficient to avoid compression if the current Code derived displacements had been applied.
- There is evidence that the available seismic gap was not large enough to prevent some stair flights being compressed and slightly damaged during the 4th September 2010 earthquake.
- The specified gap was sufficient for the shaking experienced in the 26th December earthquake.
- The 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.
- General instructions had been given after the 4th September earthquake for any cracks over a certain size to be repaired by injection of an epoxy mortar. No evidence could be found to suggest that vertical accelerations (or response of the stair over its length) experienced in the 22nd February earthquake caused the stair failure.

Recommendation

- Known alternatives to the seismic gap detail used in this building should be used on all new buildings, and for replacing the stairs in this building. These alternatives minimise significantly any likelihood of the stair collapsing because of insufficient displacement allowance.
- DBH should issue an advisory note, warning of the potential issues and lack of resilience with the gap and ledge stair detail for new and existing buildings.
- Consideration should be given to including a provision in the Building Code requiring clearances and seatings for stairs to be capable of sustaining a nominal drift of twice that estimated for the Ultimate Limit State (ULS), after allowances for construction tolerances.
- The concept that a specified seismic gap must not be compromised under any circumstances should be promoted.

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 stairs in the Forsyth Barr Building on the south-east corner of Armagh and Colombo Streets 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 severely damaged or collapsed in this earthquake.

2 Objectives and Scope

The following are the investigation's objectives and scope set by the DBH:

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

- *The original design and construction of the buildings*
- *The impact of structural alterations to the buildings*
- *How the buildings performed in the 4th September 2010 and Boxing Day 2010 earthquakes, in particular the impact of the earthquake on the building*
- *What assessments - including the issuing of green placards 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 relevant agencies 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 Forsyth Barr Building stairs, the scope of the investigation has included consideration of the following:

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

3 Approach / Methodology

3.1 General

At the commencement of our investigation, the collapsed stairs had been removed from the building and placed on an adjacent carpark to the east. We were unable to establish which stair units had come from which levels of the building. Concrete and reinforcing steel samples had already been taken from the stair units. Because of access difficulties, and the danger of the units above the collapsed ones being dislodged in a strong aftershock, we did not personally inspect the seating/landings from which the stairs fell. However, we had access to a number of photographs of these areas taken by others in the days immediately after 22nd February 2011.

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 Forsyth Barr 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 Forsyth Barr site. We commissioned boreholes to be drilled at the site of the nearest seismograph.

Some of the tenants who were in the building at the time the stairs collapsed were interviewed with an objective of trying to determine the sequence and timing of the collapse.

Our investigation concentrates on determining the likely maximum relative horizontal movement (i.e., the maximum drift) between adjacent floors in the building during the three earthquakes of interest.

While it would have been relatively easy to estimate maximum drift 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. 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.

The most sophisticated analytical tool available in these circumstances is a 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 is 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, and access to the building.

3.2 Information Gathering

The following data was available to us:

- An apparently complete set of “For Construction” drawings of the building, including stairs, dated July 1988.
- Building Manager’s engineers’ Level 2 Rapid Assessment report completed in September 2010
- The Christchurch City Council’s Property File (1988- August 2010)
- Owner’s structural engineers’ reports dated 31st March 2011 and 13th April 2011.
- Soils information from the Christchurch City Council’s Orbit database
- Soils investigations undertaken in June 2011 for the REHS seismograph site
- Photographs from several sources
- Tenants’ accounts of the collapse
- Eye witness accounts of the collapse
- Post-collapse test results for steel reinforcing bar and concrete
- Our own observations of the remaining stairs at Levels 14, 15 and 16 (see Appendix A2.6).

3.3 Structural Analysis

Analytical models of the structure have been created by Beca and time-history analyses carried out using actual records from the 4th September 2010 and 22nd February 2011 earthquakes to estimate the inter-storey drifts that occurred during each, and the time-sequencing of these.

In addition, analyses of a typical stair unit have been undertaken to:

a) determine the effect of vertical accelerations, and

b) understand its failure mode if it were to be placed under compression.

3.4 Geotechnical Assessment

The available geotechnical data for the site was interpreted and conclusions drawn on the likely effect of the performance of the site on the stair collapse.

3.5 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 Description

The eighteen-storey building, designed in 1988, is founded on a shallow raft, and its lateral resilience is provided by the frame action of the reinforced concrete beams and columns. For three levels above the ground level, the floors extend beyond the footprint of the tower to form a podium on the south and east sides. Refer to Figure 4.1 which shows a photograph of the building taken from the east on Armagh Street.

The building system is unusual in that it essentially comprises two triangular, framed portions which are linked together by a precast concrete floor system. Figure 4.2 shows a typical floor plan and indicates some of the building features obtained from the drawings available to us.

The stairs are orientated diagonally within the tower in a north-east/south-west direction. Figure 4.2 locates the position of the stair. The stairs are of the "scissor" type. In a scissor stair, two stairs are provided within a single shaft and are separated by a light weight partition running in-between and over the full height of the stair. Access is achieved by winding down the stairwell, swapping sides at each floor, and passing under the stair flight of the other stair. Figure 4.3 shows the general arrangement. One of the risks associated with such a stair system is that, if a flight collapses on one side of the shaft, it will potentially render impassable both stairs.

The majority of the stair flights were precast concrete units (cast as a single unit, not as separate flights connected by a cast in-situ slab as shown on the drawings) cast into the supporting concrete beam at the top landing (refer to Figure 4.4 for the detail). At their lower end, they were seated on a steel channel with a horizontal gap specified to be 30 mm wide (refer to Figure 4.5 for the detail). The steel channels also support the 'toilet' slab.

The building was originally called Robert Jones House, and was constructed at about the same time as the similarly-sized PriceWaterhouseCooper building a few metres further down Armagh Street.

A building designed in 1988 could be expected to perform to essentially the same level as a similar building in 2010.

Figure 4.1 : The Forsyth Barr Building from Armagh Street (looking south-east)

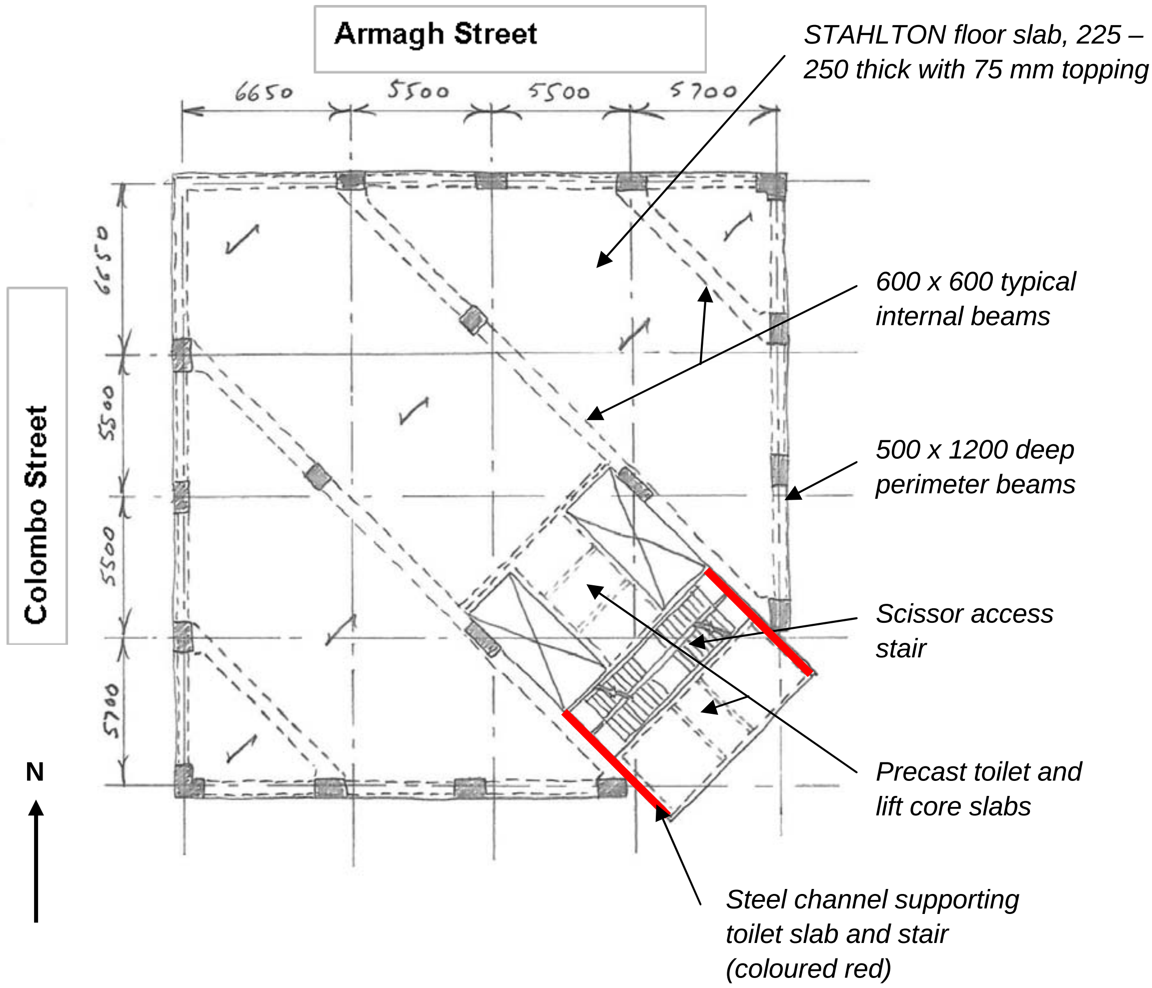

Figure 4.2 : Typical Floor Plan

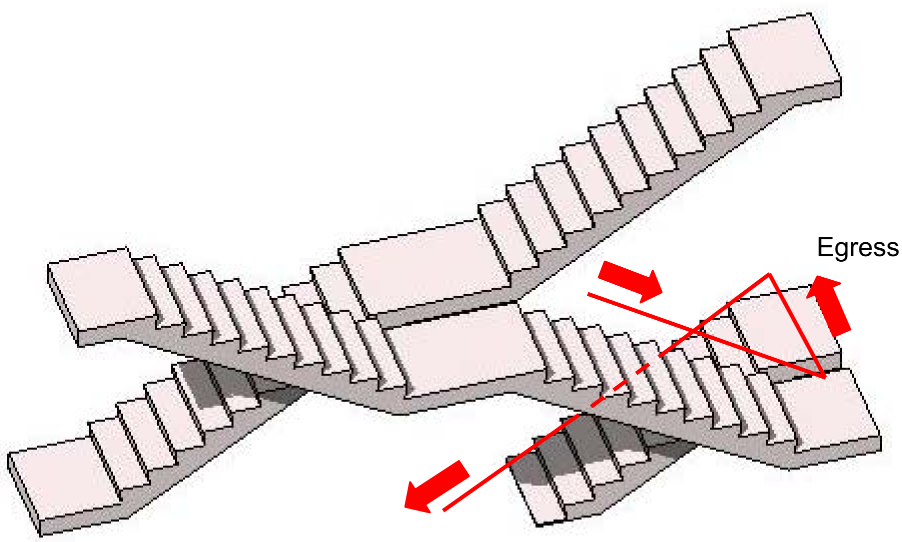

Figure 4.3 : 3D View of Typical Scissor Stair System between Two Adjacent Levels

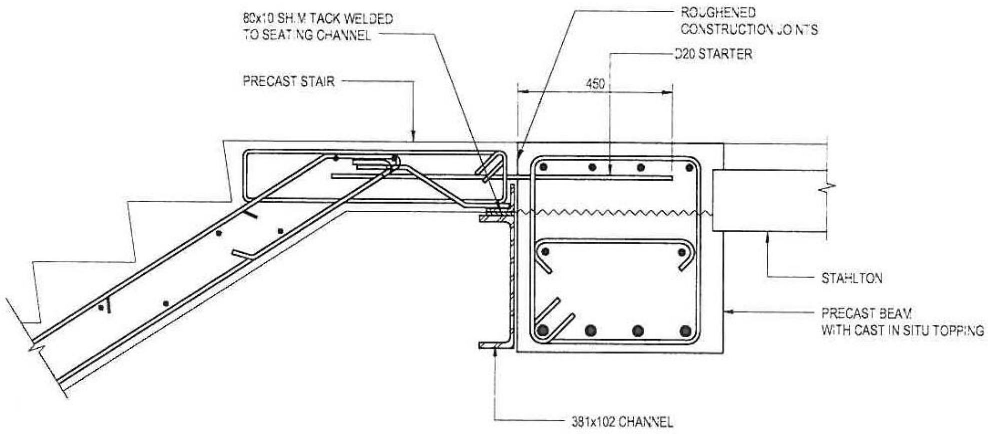

Figure 4.4 : Typical Stair Top Support Detail

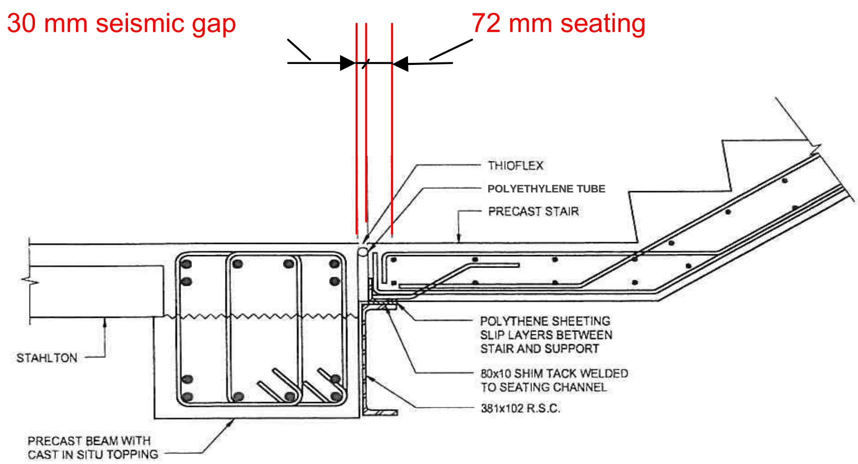

Figure 4.5 : Typical Stair Bottom Support Detail

4.2 Building History

Table 4.1 summarises the major events in the building's life.

Table 4.1 - Major Changes to Building During its Life

Date	Event	Comment
1988	Designed as Robert Jones House	
1988	Constructed	Building Consent 1988
5th Sept. 2010	Level 1 Rapid Assessment	Unsafe (Red)
5th September 2010	Level 2 Rapid Assessment	Restricted Access (Yellow)
6th September 2010	Level 2 Rapid Assessment after inspection of stairs and propping completed	Inspected (Green)
15th September 2010	Level 2 Rapid Assessment after inspection of in-filled slab at floor level 7.	Confirmed as Inspected (Green)
October – February 2011	Repairs to floor coverings on stairs and landings	
November – February 2011	Repairs to structural elements	

4.3 Site Investigations (Soils, Seismology)

4.3.1 Building Location

The building is located at 764 Colombo Street, Christchurch, on the south-east corner of Colombo and Armagh Streets. Figure 4.6 shows the building and the general locale.

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Figure 4.6 : Location of Building Site

4.3.2 Information Sources

Soils

The site and surrounding area was inspected by a Beca senior geotechnical engineer on 5th April 2011 as part of this investigation. The Orbit database was interrogated and summary borelogs have been obtained and reviewed for the vicinity of the Forsyth Barr building, and for strong-motion recording sites REHS, CCCC, CBGS and CHHC which surround the CBD. This has allowed a preliminary assessment to be made of ground conditions and hence the likely behaviour under seismic shaking. However, the borelogs are summaries, and do not contain strength or consistency information.

The following investigations have been made by others at the time of design of the building:

- Three machine boreholes to a depth of 20 m with standard penetration testing at regular intervals, located at the south-west, north-east and south-east corners of the building, just beyond the footprint.
- Two machine bores to a depth of 14 m with no standard penetration testing, located within the tower footprint at the south-west corner.
- Geotechnical investigation report dated April 1988 by Soils and Foundations Ltd.

Boreholes were undertaken by Beca 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 A1.3.

Seismology

The nearest permanent seismograph to the Forsyth Barr building is at the Resthaven Rest Home (REHS) in Colombo Street, about 100 metres south of Bealey Avenue. This is about 800 metres to the north of the Forsyth Barr building site. The next closest permanent seismographs were in the Botanic Gardens (BGS, 1.4 km W), near the Christchurch Hospital car parks (CHHC, 1.0 km SW), and near the Catholic Cathedral College in Barbadoes Street (CCCC, 1.3 km SE). Temporary seismographs were installed in the Christchurch Police Station after the 4th September earthquake.

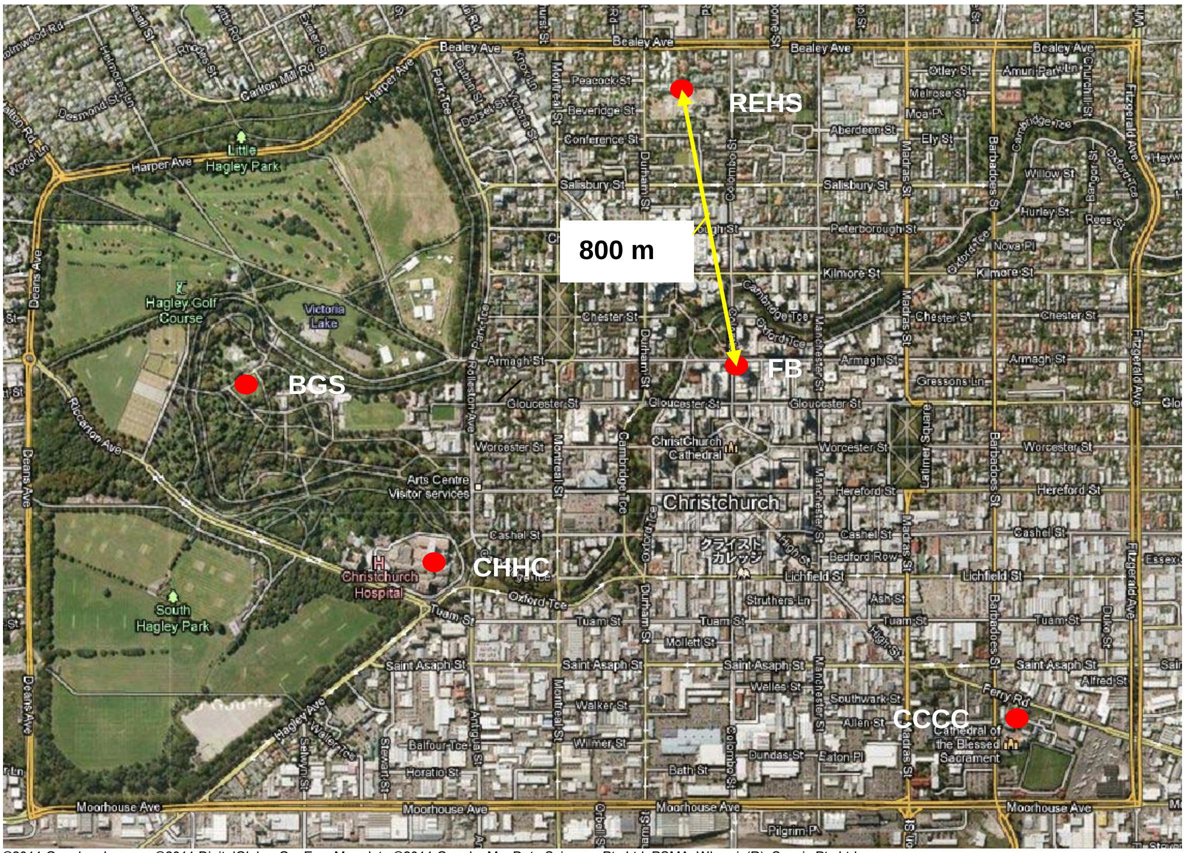

Figure 4.7 : Location of Building Site Relative to Strong-Motion Recording Sites

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 for instrument characteristics. Information on the soil conditions beneath each station were sought from GNS Science (operators of GeoNet) in order to see whether they were similar to those underneath the Forsyth Barr 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 Victoria Square 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.

4.3.3 Interpretations

Soils

From the Orbit Database:

Bores 5820 and 5821 were put down in 1987, presumably to inform the foundation design for the building. Bore 5222 is also indicated on the database, but the log is not available. These bores identified a near-surface fine to medium sand (which was evident in an open trench on Armagh Street during our site inspection) extending to around 2.5 m depth, underlain by a sandy gravel to a total of around 10 m depth. The sandy gravel was found to be underlain by typically fine to medium sand, with varying but typically rare organics. Artesian water pressures were reported, and a nearby bore (1924, located a short distance east on Armagh St) identified artesian water pressures in gravel layers from around 25 m depth.

CHHC - bore 8542 indicates around 20 m of clayey soils near the surface, so is significantly different to the profile at the Forsyth Barr site.

REHS - bores 2140 to 2142 directly across Colombo St from this site indicate near surface sands to around 2 m depth overlying around 6 m of peat or clayey soils over sand and gravel, again significantly different to the profile at the Forsyth Barr site.

CCCC - bore 2123 indicates clay and sand to 22 m depth, with no closer description of any specific subdivisions.

From Investigations at Site Undertaken by Beca in June 2011:

Liquefaction ejecta was evident around the perimeter of the building during our site inspection on 5 April 2011, and fine sand had found its way into the ground floor shops. Footpath levels appeared to have dropped slightly relative to the structure. No evidence of liquefaction was observed in the rear car park to the south-east.

Based on this information, it seems likely the liquefaction ejecta evident on the surface originated from the near surface sands. Deeper sands are considered unlikely to have migrated up through the intervening sandy gravel. The apparent ground settlement is also likely to have occurred in this surface sand layer. The liquefaction resistance of the sandy gravel layer depends on its grading and relative density, but it seems likely to have performed relatively well. The deeper sands, lying close to the artesian aquifer, are more likely to have liquefied in the course of the 22nd February earthquake.

Having also carried out further soils investigations at the REHS recording site in June 2011, we reviewed the existing geotechnical investigation data at the Armagh/Colombo Streets site and have concluded:

- The near surface soils are predominantly silty sands, and these extend to depths of between 1.5 and 3.0 m. Below these, the site is underlain by medium dense sandy gravels to a depth of around 10 m. These gravels are underlain by dense to very dense sands. The sands are underlain by a thin silt aquiclude, with water bearing sands and gravels below that.

- Ground water levels were reported at around 2.5 m below ground level.

Assessment of Ground Performance:

Analysis of the Scala Penetrometer Test (SPT) profiles from the three bores put down around the building indicates liquefaction potential in the upper sandy gravels beyond around a peak ground acceleration (PGA) of 0.3 g. It is commonly accepted that gravels can liquefy, though there is little data on the difference in liquefaction resistance between sands and gravels. The performance of the Forsyth Barr building in the 22nd February event suggests that liquefaction of the sandy gravels, if it did occur, was not extensive. Analysis indicates that liquefaction of the underlying sands is unlikely up to and beyond a PGA of 0.8 g. The near-surface sands largely lie above ground water level, so are not likely to have liquefied. They may well have settled as a result of the shaking, explaining the observed 50 mm or so difference in level between the building and the footpath. Some silty sand was evident within the ground level shops. It is postulated that this was derived from the near-surface sands, transported by water driven from beneath the building by shaking-induced settlement of the medium-dense sandy gravels. Our analyses indicate that around 50 mm of shaking-induced settlement may have occurred in the medium-dense sandy gravels.

Foundation/Subgrade Parameters for Structural Analysis

The Forsyth Barr tower is supported on a reinforced raft slab at a depth of around 2.5 m below the ground floor level. The raft sits on the medium-dense sandy gravels, or on compacted hardfill on the sandy gravels. The edge of the raft is calculated to have a geotechnical ultimate bearing capacity of over 1000 kPa under vertical loading, assuming it is not affected by liquefaction.

The spring stiffness under the foundations for use in analysis depends on the magnitude of loading, and the following points on a curve are considered reasonable:

100 kPa applied load – 3 mm deflection

200 kPa applied load – 9 mm deflection

500 kPa applied load – 30 mm deflection

The podium is founded on spread foundations above the medium-dense sands and gravels. The stiffness values given above can be used as a first approximation.

We have not seen any evidence that the foundations have moved laterally relative to the ground (i.e., no gapping or shoving). This means that the estimated base shear that occurred during the earthquake, associated with (say) a 25 mm elastic displacement, presents a lower-bound stiffness.

Strong-Motion Recording Sites

There was very little definitive soils information available for these sites, and the soil profiles were initially inferred from the information collected from the Orbit data base.

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 m. The pug is underlain by sand or sandy gravels. The profile is therefore different to that at the Forsyth Barr 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 different to that at the Forsyth Barr building site. Our investigations (June, 2011) at the REHS site comprised a machine bore to a depth of 15 m with standard penetration testing at regular intervals, and a cone penetration test to 20 m. The investigations identified a near-surface gravel layer extending to a depth of 1.3 m, which is underlain by typically firm and commonly organic silt to around 9 m. Characteristic shear strengths of the silt (derived from the CPT results) range from 10-15 kPa in a 2 m thick soft peat/organic silt at a depth of around 5 m 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 22 m depth, 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 Forsyth Barr 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, proximity to areas of liquefaction in September 2010 and February 2011, and (at two sites) SPAC (Spatial Autocorrelation, a micro-tremor technique).

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 the stairs in this building.

4.4 Design, Drawings and Specifications

The following building data was available to us:

- An (apparently complete) set of structural drawings including details of the stairs, dated 1988
- The Christchurch City Council's Property File (1988- August 2010)
- Original geotechnical investigations carried out for the building.

4.5 Variations during Construction

No evidence has been found which would indicate that the building frame was constructed differently from the drawings.

Each scissor stair flight is shown on the drawings as being comprised of two precast units connected together at the mid-height landing with a cast in-situ concrete infill. Observations of the collapsed flights indicate that the flights were precast as a single, full-length unit. It could be expected that tight control would have been kept on the overall length of the precast unit, but pre-casting may have removed the option of adjusting the stair flight length on site should this have been required to maintain the seismic gap. There is possible evidence that the adjacent floor topping at the bottom landing may have been poured against polystyrene sheet in the seismic gap. Polystyrene was seen in photos to have been left in at least some of the gaps. This could have ensured the gap between the concrete faces was correct, but would have potentially reduced the available clearances as polystyrene is not infinitely comprehensible.

At least two of the collapsed stair units inspected after removal, and two still in place, appeared to have had their lower landing edges ground back – presumably at time of construction (refer to Figure 4.8) - but this could not be substantiated. The nature of the grinding (i.e., not over the full width of the stair landing in all cases) would suggest that it was done with the stair in place. It is also apparent that the grinding does not extend through the steel armouring on the bottom corner of the landing. This has potential implications for the available clearance in the seismic gap that will be discussed later in this report.

4.6 Post-Occupancy Alterations

No post-occupancy alterations that would have an impact on the performance of the stairs were identified (because the collapse and demolition majorly changed the pre-collapse situation). We have been unable to determine whether the grinding back of the ends of the landings and any filling/grouting of the seismic gap were carried out as part of refurbishment works in the stair well.

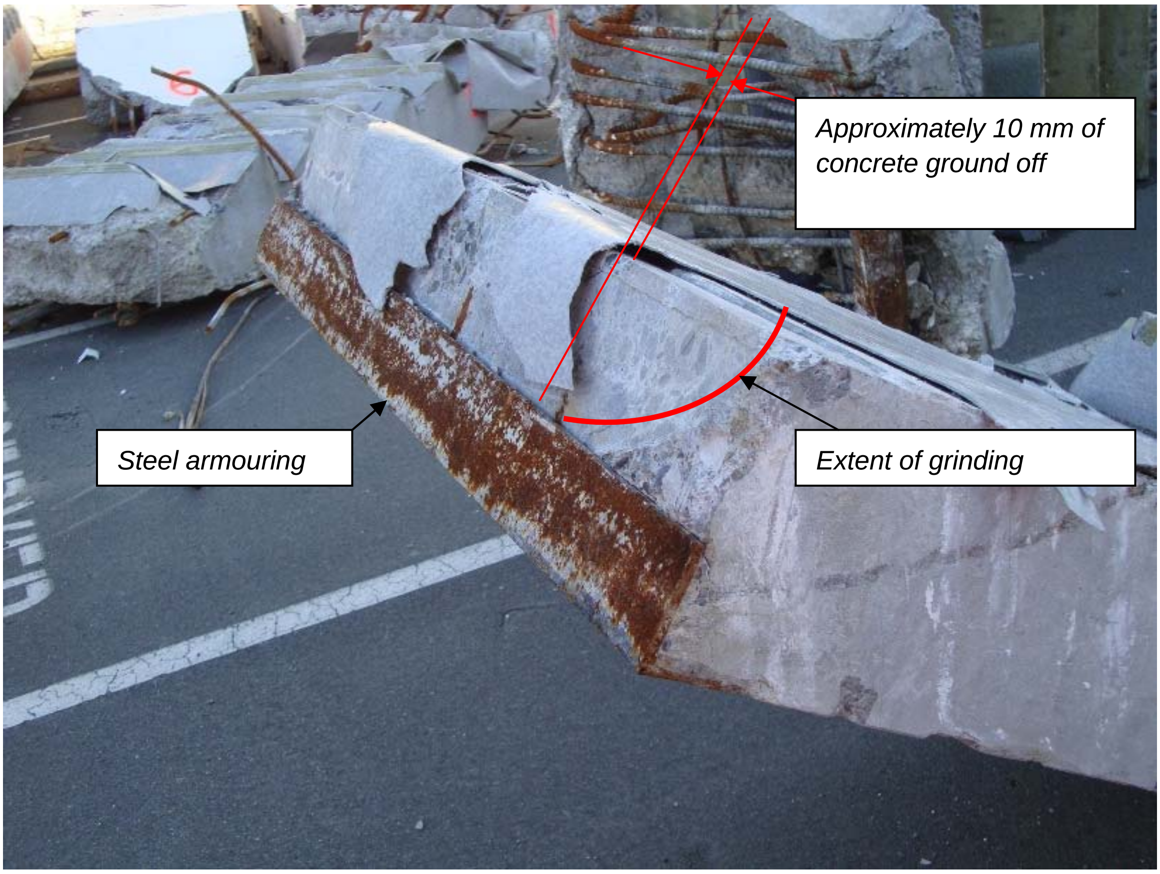

Figure 4.8 : Ground-Back End of Landing in Collapsed Stair Unit

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, 1.4 km to W)
- Cathedral College (CCCC, 1.3 km to SE)
- Christchurch Hospital (CHHC, 1.0 km to SW)
- Resthaven Rest Home, Colombo Street North (REHS, 800 m to N)

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 all four buildings being investigated by DBH). GNS Science has re-computed the CCCC recordings to make equivalent N-S and E-W components.

5.1.2 Acceleration vs Displacement Spectra

Response spectra are a convenient way of estimating 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 (T = 0.5 to 1.0 sec). 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 or as a displacement) 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.

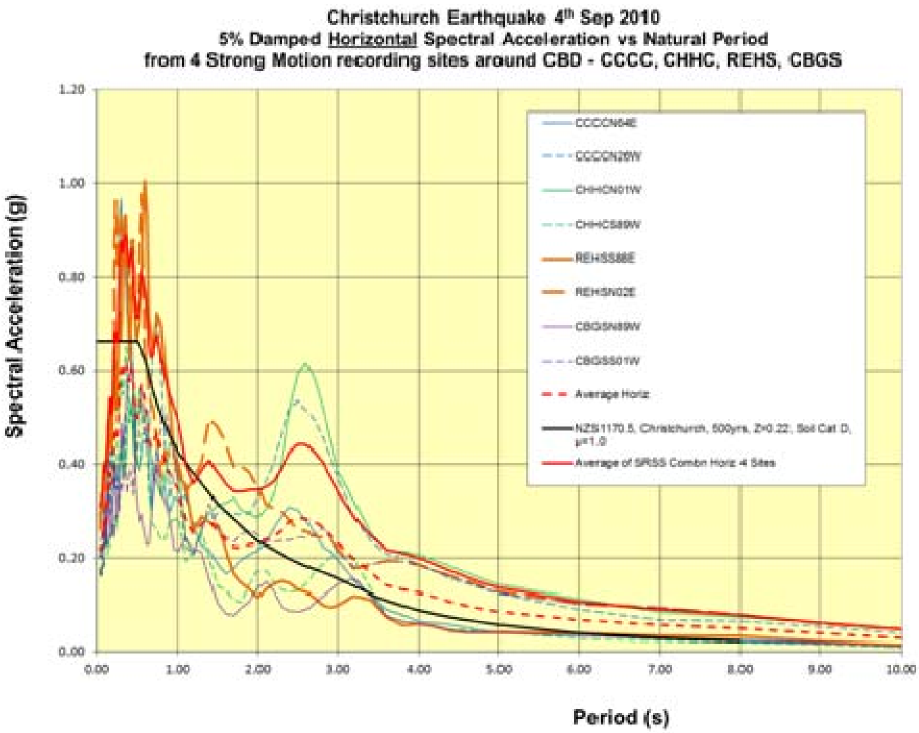

Figure 5.1 : Traditional Acceleration Response Spectra from Recordings of the 4th September 2010 Earthquake (5% Damping)

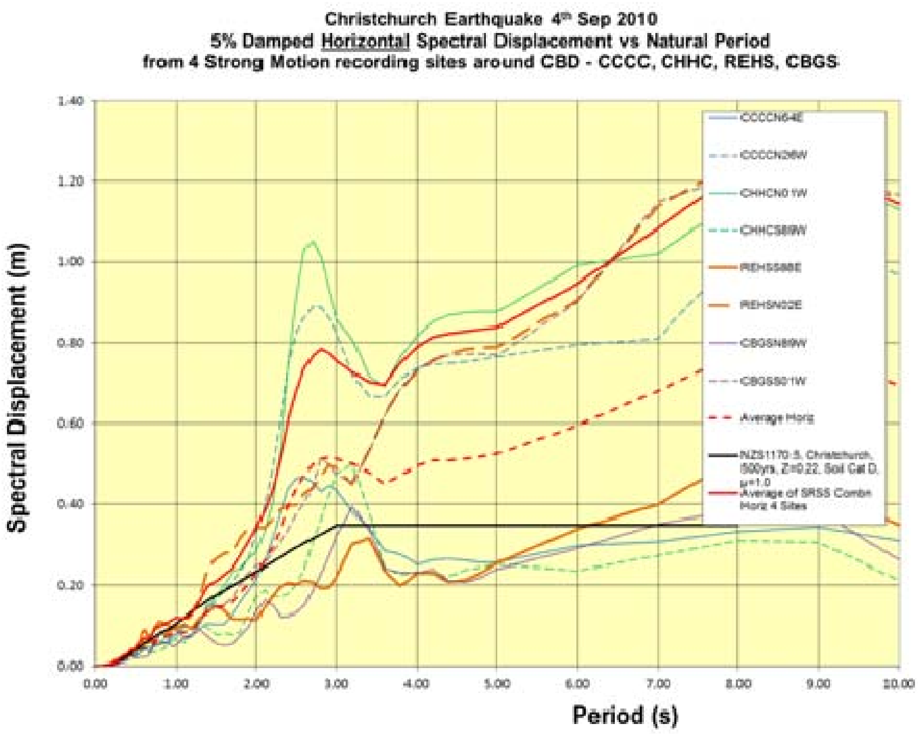

Figure 5.2 : Traditional Displacement Response Spectra from Recordings of the 4th September 2010 Earthquake (5% Damping)

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 the relative horizontal movements between storeys of the Forsyth Barr building. These spectra for the 4th September 2010 earthquake are shown in Figures 5.3.and 5.5.

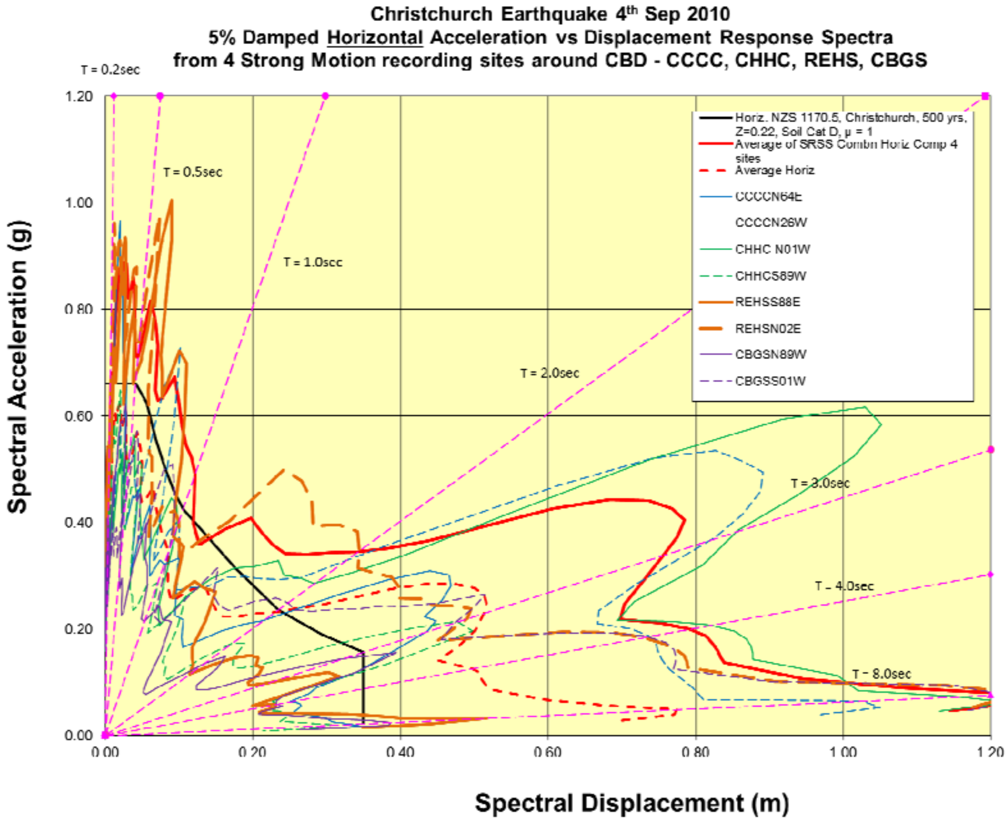

Figure 5.3 : Horizontal Acceleration-vs-Displacement Response Spectra from Recordings of the 4th September 2010 Earthquake (5 % Damping)

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 from the origin for natural periods of 0.2, 0.5, 1, 2, 3, 4 and 8 seconds are shown (rotating clockwise, respectively).

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 on a Saturday morning 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):

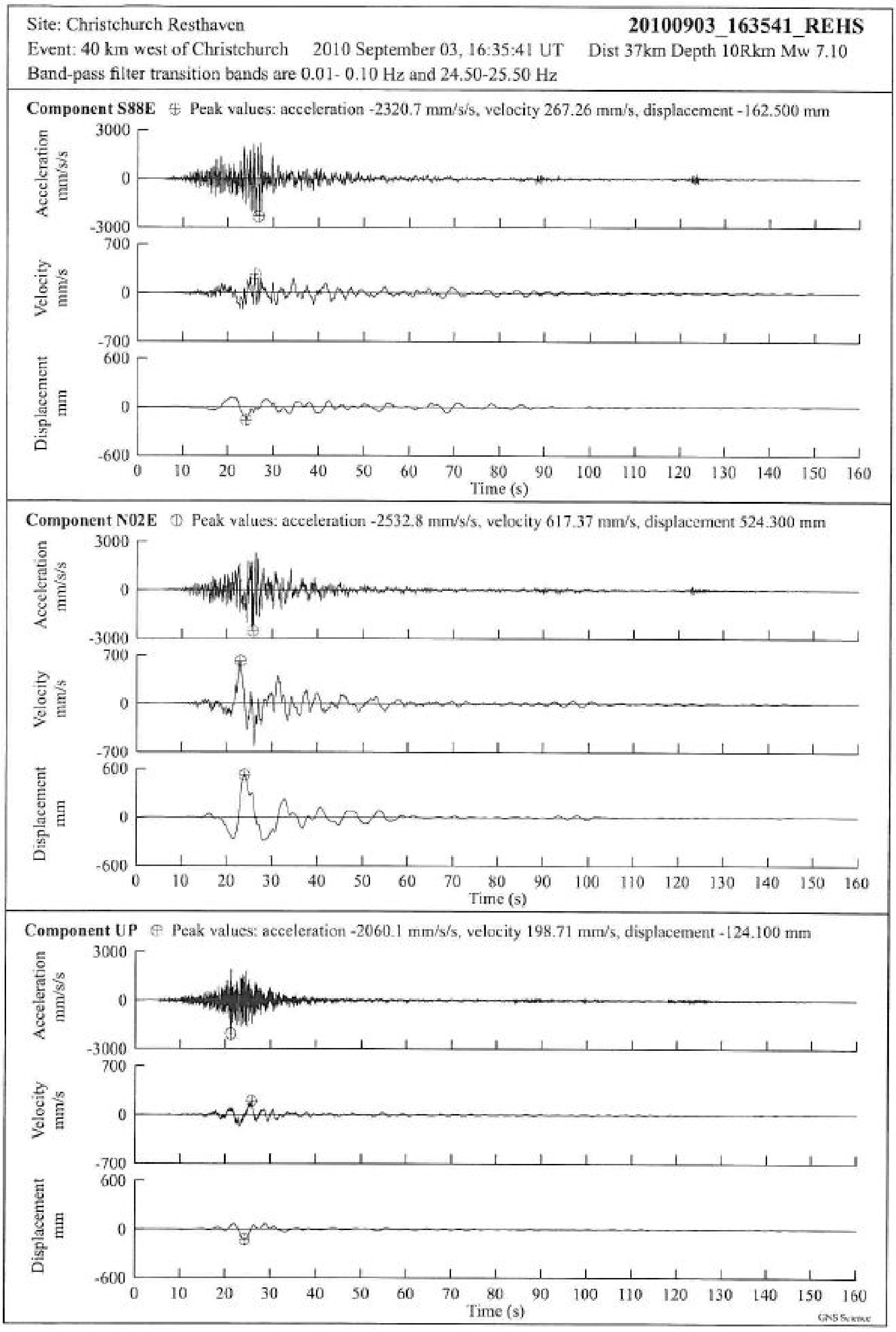

Figure 5.4 : Acceleration, Velocity and Displacement Records from the REHS site

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:

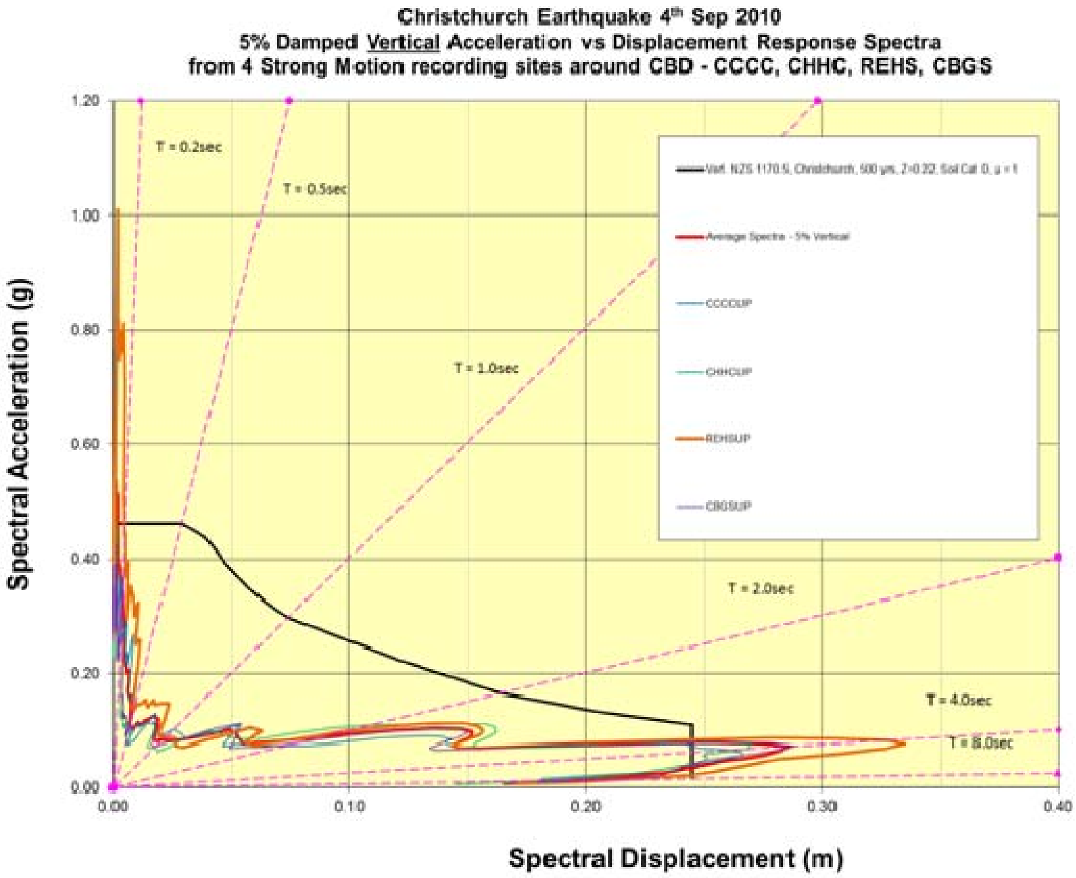

Figure 5.5 : Vertical Acceleration-vs-Displacement Response Spectra from Recordings of the 4th September 2010 Earthquake (5 % Damping)

5.2.2 Observed Building Performance

Minor structural damage was observed after the 4th September 2010 earthquake, including some cracking and deformation of a few flights of stairs. The Level 1 rapid assessment undertaken within a few days of the earthquake under the authority of Civil Defence resulted in the building being initially placarded Red (Unsafe). This was later revised by the Level 2 assessment undertaken by Beca (the property manager's engineer), first to Yellow (Restricted Access) and, following further investigation of the stairs (breaking open of the timber-framed bulkheads under the worst flights and checking the seating of the lower landings - which indicated no significant movement) and propping of a vehicle ramp in the podium, to Green (Inspected). The assessments are reproduced in Appendix A2.

Subsequently, the owner's structural engineers undertook an inspection of the building, and prepared instructions in November 2010 for the repair of cracked structural elements. The engineers have advised that they were told to exclude the stairs from consideration. The instructions were general ones for epoxy grout injection relating to the crack width in all structural elements.

Building occupants interviewed have stated that repairs to earthquake damage to floor coverings on the stairs were underway in the period between the September 2010 and February 2011 earthquakes.

5.2.3 Calculated Building Performance

An analytical model of the structure was developed by Beca, and time-history analyses using actual records of the earthquakes have been completed to estimate the extent of the inter-storey drifts and the way in which they varied during the earthquake on 4th September 2010. The analyses carried out are described in Appendix A3. A comparison between the spectra from this earthquake and that obtained from the Boxing Day 2010 earthquake indicates that the effects of the latter event would have been insignificant for this building. Therefore, this event was not investigated in detail.

The reports of damage of the building structural frame indicate that it suffered only minor damage during the September earthquake. The time-history analyses were therefore completed using an elastic model of the building structure, with the flexibility of the elements increased to allow for cracking.

A plot of the inter-storey drift (relative horizontal displacement between adjacent levels) with time for Levels 13 to 14, for the 4th September earthquake, is shown in Figure 5.6. The drift between these two levels was the largest in the building of the analyses that were carried out. The drift shown was calculated at the stair location for the direction parallel to the stair flights.

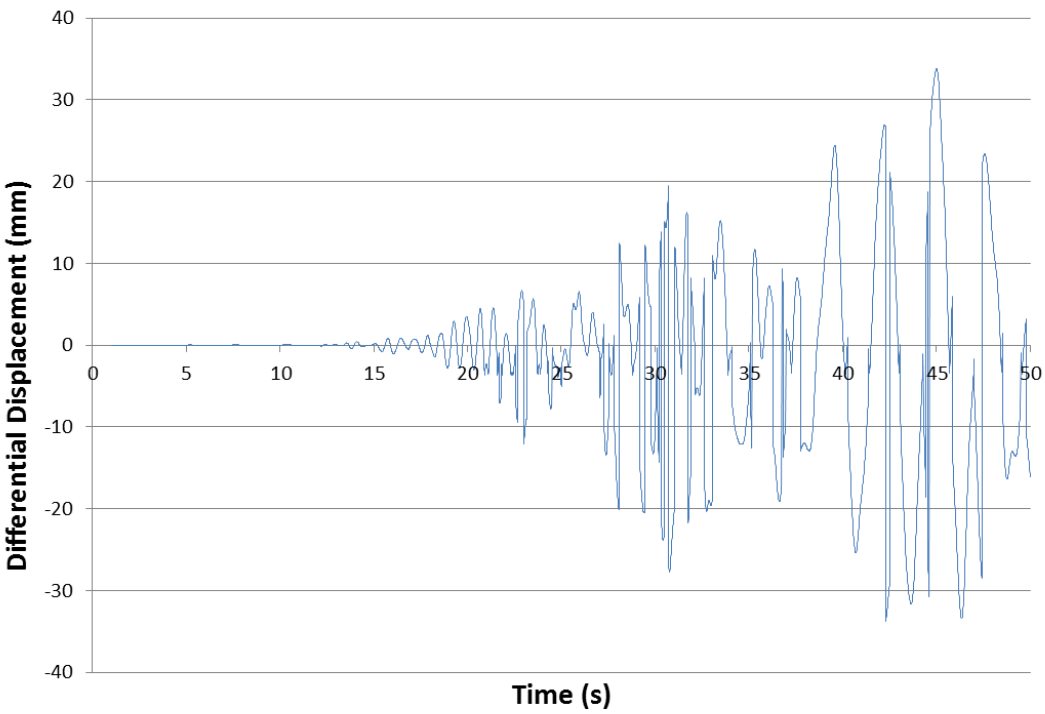

Figure 5.6 : Predicted Variation with Time of the Inter-storey Drift between Levels 13 and 14 during the 4th September 2010 Earthquake Obtained from Time-History Analyses Using the REHS Strong-Motion Record

5.3 Boxing Day, 26th December 2010

5.3.1 Earthquake Records

An indication of the duration of strong shaking during this earthquake can be seen (Figure 5.7) from GeoNet's record from the Catherdral College instrument CCCC which was closest to the epicentre (focal depth 12 km) of this Magnitude 4.9 event. The epicentre was about 4.5 km from the Forsyth Barr building.

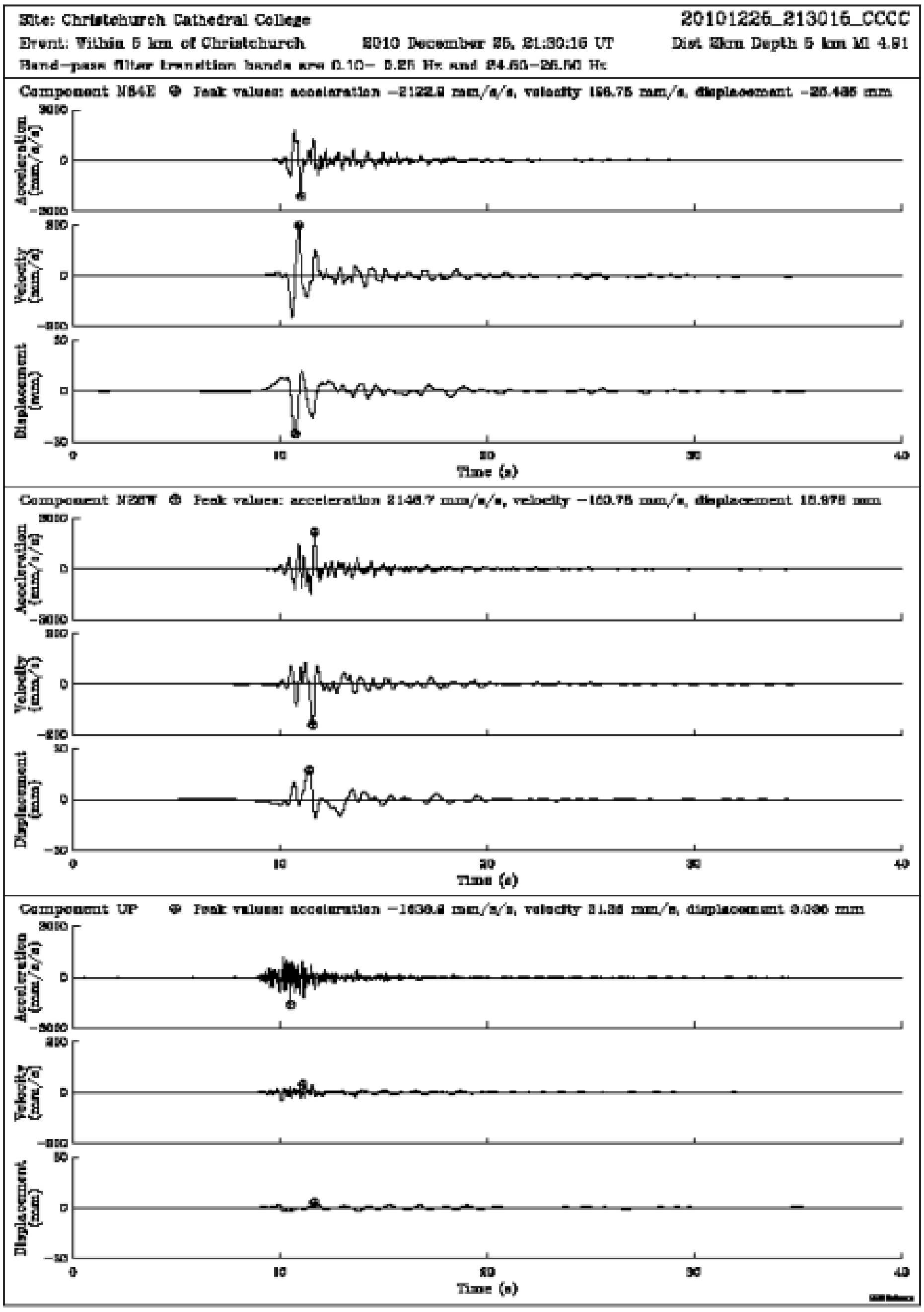

Figure 5.7 : Boxing Day Acceleration, Velocity and Displacement Records from the CCCC site.

Figures 5.8 and 5.9 below are Acceleration-vs-Displacement response spectra for this earthquake:

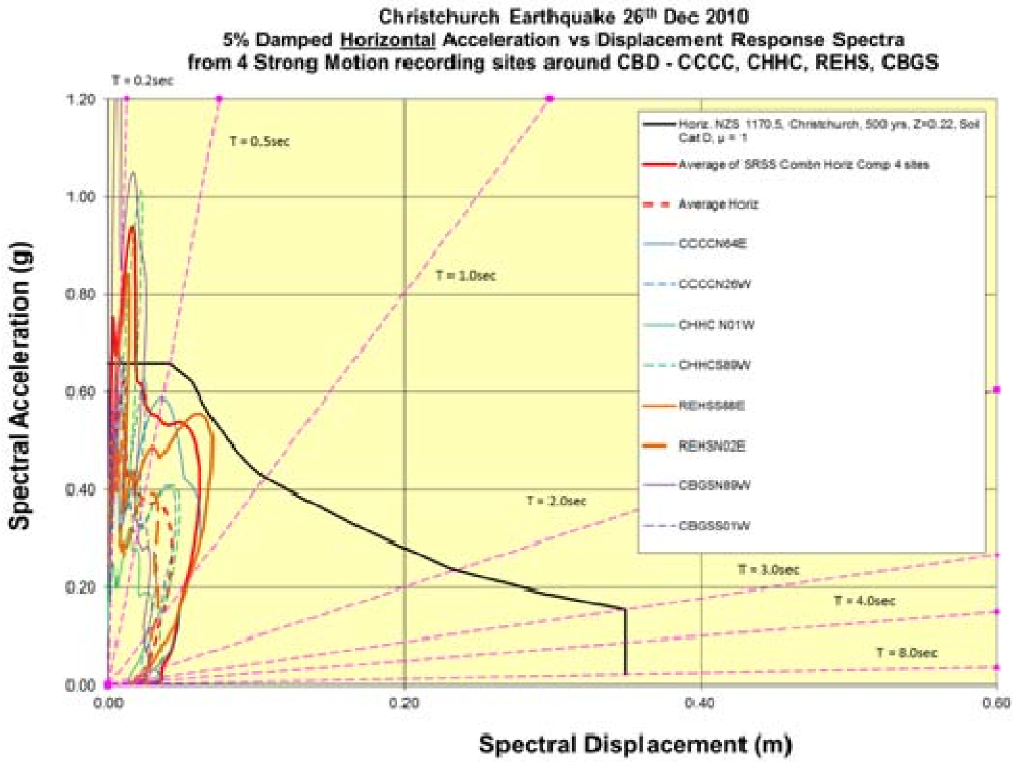

Figure 5.8 : Horizontal Acceleration-vs-Displacement Response Spectra on 26th December 2010 (5 % damping)

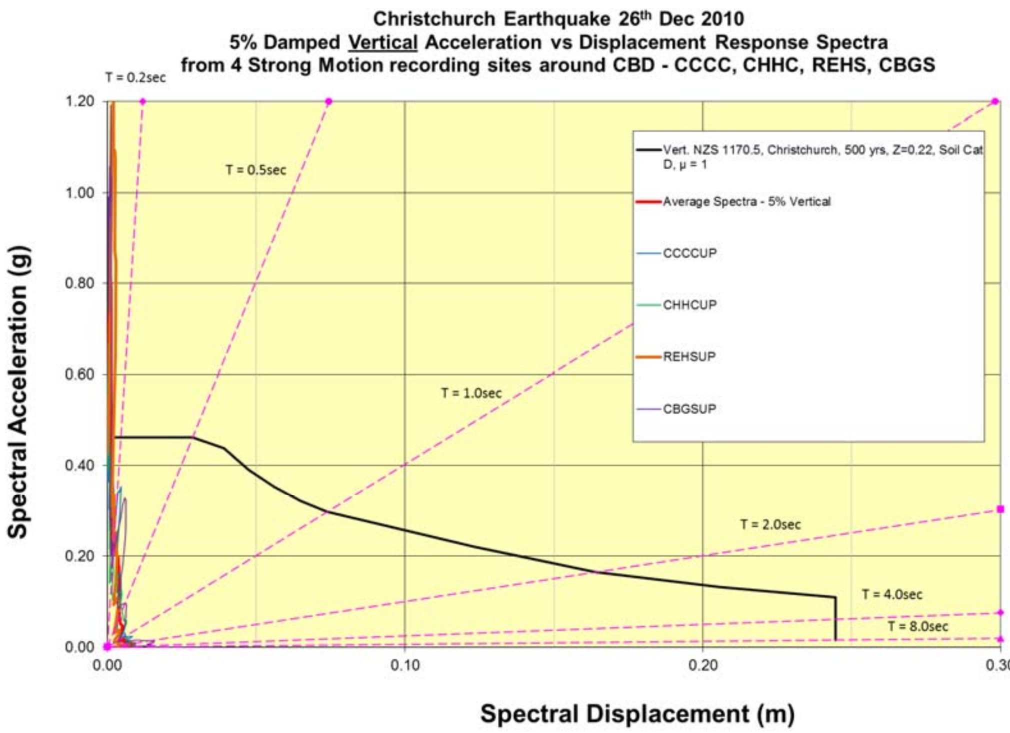

Figure 5.9 : Vertical Response Spectra from Recordings on 26th December 2010 (5 % damping)

5.3.2 Observed Building Performance

No reports of additional structural damage to the Forsyth Barr building from the Boxing Day 2010 earthquake have been sighted.

6 Effects of 22nd February 2011 Event at Forsyth Barr Site

6.1 Earthquake Records

This Magnitude 6.3 earthquake occurred at 12.51 pm, and its epicentre was approximately 10 km south-east of the building site, at a focal depth of 5 km. An indication of the duration of strong shaking can be seen from GeoNet instrument REHS about 800 m to the north of the building site.

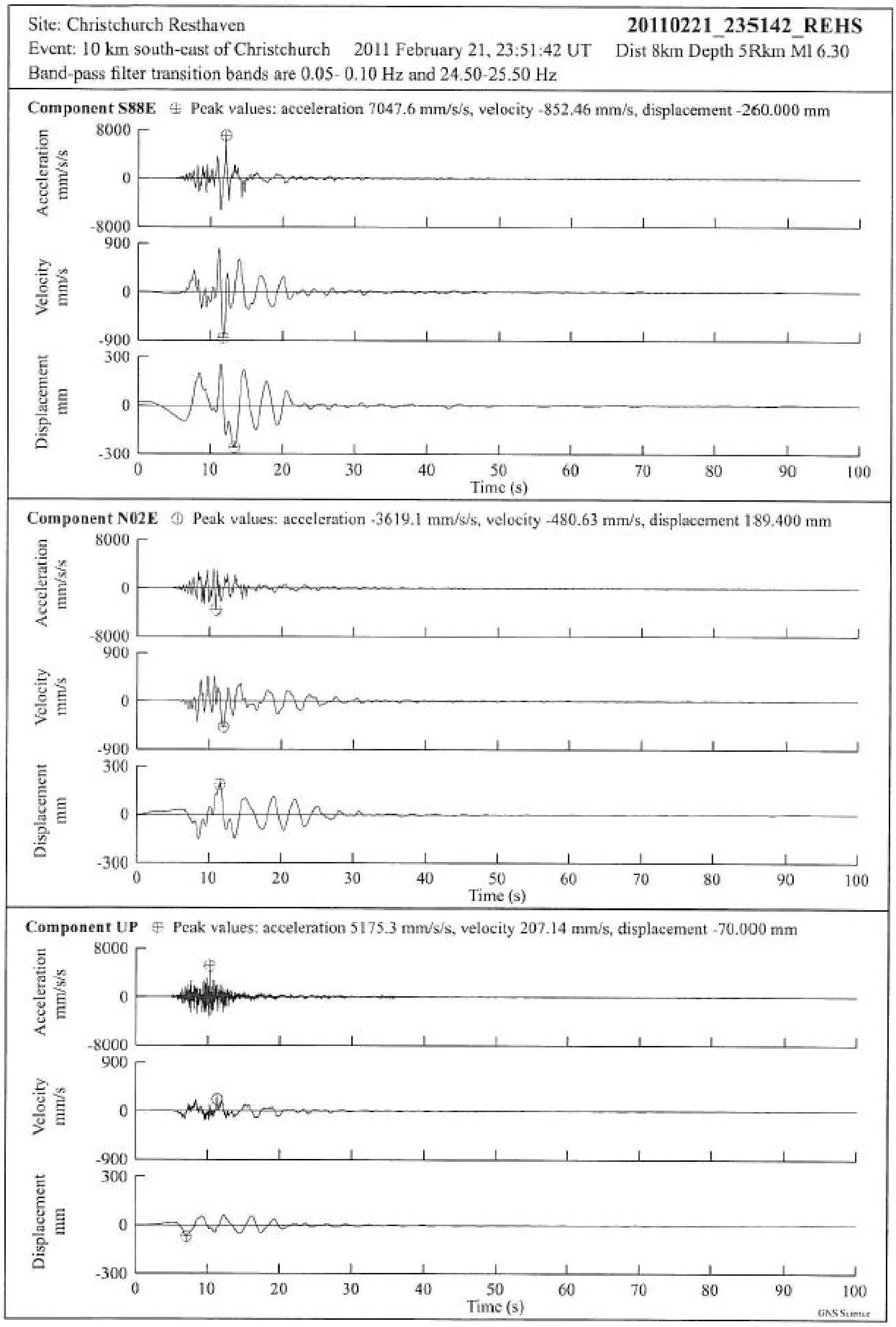

Figure 6.1 : Acceleration, Velocity and Displacement Records from the REHS site

Figures 6.2 and 6.3 below are Acceleration-vs-Displacement response spectra for this earthquake:

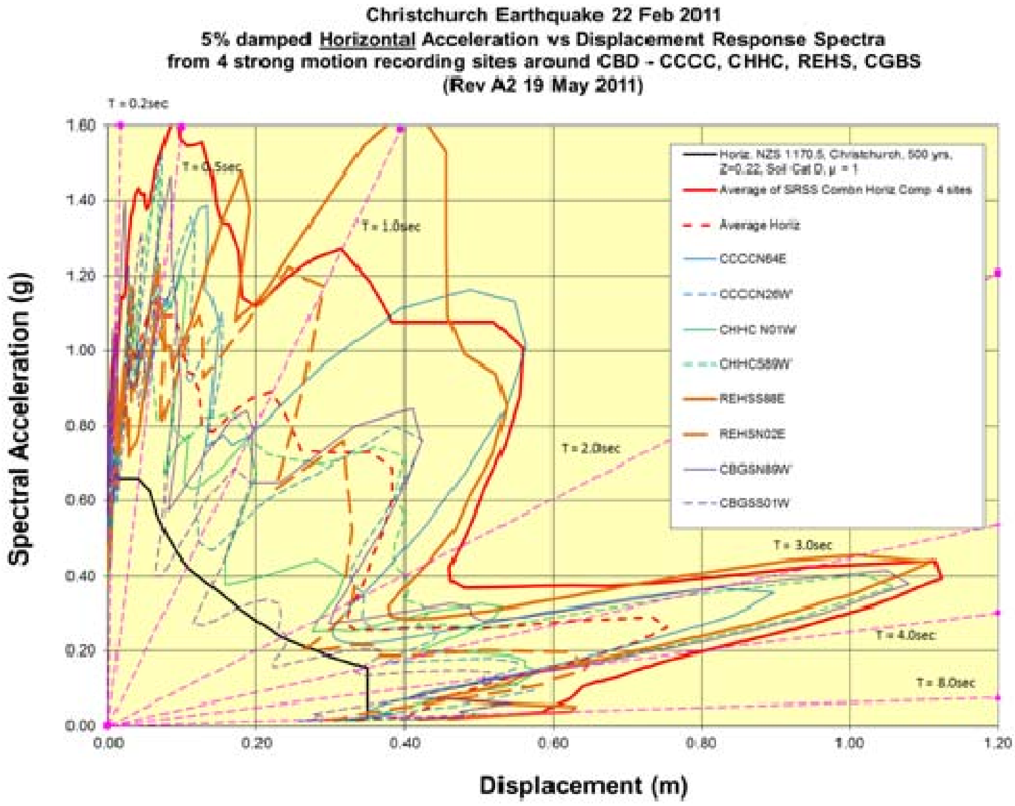

Figure 6.2 : Horizontal Acceleration-vs-Displacement Response Spectra from Recordings on 22nd February 2011 (5 % damping)

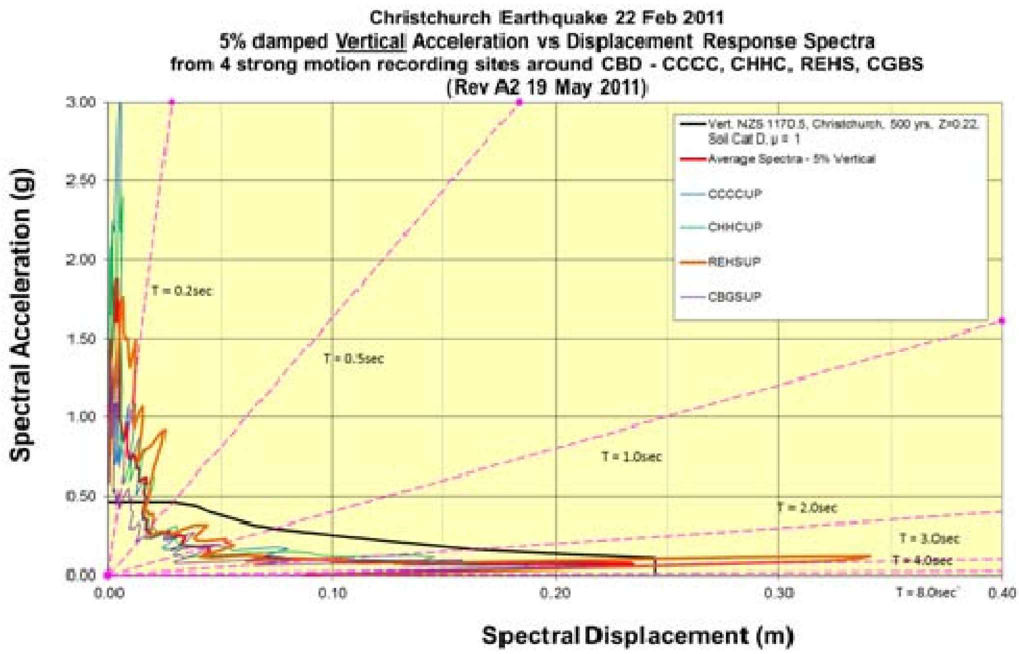

Figure 6.3 : Vertical Acceleration-vs-Displacement Response Spectra from Recordings on 22nd February 2011 (5 % damping)

6.2 Observed Building/Site Performance

6.2.1 Stair Performance

In the 22nd February 2011 earthquake, the stair collapsed removing the means of escape from the building after the earthquake.

Urban Search and Rescue (USAR) personnel indicated that, when they gained access to the building, they observed that the stair flights had collapsed up to Level 13 on one side of the stair well, and up to Level 15 on the other. Subsequent reports prepared by the owner's engineer record that the *stairs have collapsed up to Level 15, from Level 15 to 16 the north stair remains*. *This stair has approximately 20-25 mm seating remaining. The stairs above have 40 mm plus seating remaining*.

A team from USAR removed the collapsed stairs after breaking them in half at their intermediate landings. The removed stairs have been observed in the carpark to the east of the building. Evidence of cutting/grinding of the lower ends of at least two units (presumably to increase the in-place seismic gap) has been seen.

6.2.2 Building Performance

We have been unable to gain access to the building to inspect the building structure since the 22nd February 2011 earthquake. We have, however, received two reports prepared by the owner's engineer which describe the status of the building as they observed it in the immediate weeks after the earthquake.

The observations made in these reports are summarised as follows:

- Cracks up to 0.5 mm in the moment-resisting frame. These were *most prolific on Levels four to seven*.
- Spalling of concrete to the base of the corner columns.
- Up to 4 mm cracks to the floors around the corner columns, and
- Up to 2 mm cracks around other columns.
- Very little damage to diaphragms generally.
- Damage to the tie between the podium and main structure.
- Failure of a podium column.
- Cracks in the podium columns in similar locations to those that occurred during the September 2010 earthquake.
- The façade appeared undamaged except for panels removed to allow the rescue of tenants.
- Cracks less than 1 mm in the fourth-floor tower beams.

We conclude from these observations that the damage to the building's primary structure after this earthquake was minor-to-moderate.

6.2.3 Measurement of Building Residual Deformation

In early April 2011, a training exercise in the use of laser scanning from ground-based instruments was carried out on the north and west faces of the Forsyth Barr building by the technology suppliers Trimble. Preliminary analysis of the information was provided via the Ministry of Civil Defence and Emergency Management.

This information was sought because it might give an indication of any significant permanent displacements of the structure in the 22nd February 2011 earthquake that could be related to the inter-storey horizontal displacements experienced during the earthquake. The precise location of the façades before the earthquake is not known.

The accuracy of the measurement at any one point is stated to be ± 12 mm.

The preliminary analysis concluded there is little evidence of residual out-of-plane deformation to the building, with the faces of the building being within 30 mm of a best-fit plane. The building deviates from the vertical plane by 0.11 m in a northern direction, and 0.04 m in a western direction. Scanning was undertaken of the eastern face, but analysis had not been completed.

Plots of deviation of horizontal position with height from the best-fit plane do not show abrupt changes that might be considered significant.

In an interview with one of the foremen of the original construction team, he noted that the specialist subcontractor undertaking the façade installation measured the trueness of the facades and commented very favourably on their dimensional tolerance – because this was a necessary requirement for this particular façade system.

We have not considered it necessary to pursue this scanning data further.

6.3 Site Performance

The possibility of liquefaction adjacent to this building and/or site settlement during this earthquake cannot be ruled out, but is not considered significant with respect to the collapses of the stairs that occurred.

7 Collapse Description

All failed stair units had been removed from the stairwell before we were engaged to carry out this investigation. We have also been unable to gain access to the building since the 22nd February 2011 earthquake, and therefore have been unable to inspect, first-hand, the remaining stair units and the supports of the failed units. The comments and observations given below are therefore necessarily based on the observations of others who have relayed these to us.

The units that had been removed from the building were available for inspection/testing (refer to Figure 7.1) but it was impossible to judge from which levels they had come. It was also difficult to judge whether or not the damage to these flights caused the collapse or was a result of impact from falling or being hit by other stair units.

Figure 7.1 : Collapsed Stair Units Removed from Stairwell

The following observations have been made regarding the collapse of the stairs:

- The starter bar reinforcement in the top connection of a number of the stair units fractured (refer to Figure 7.2) and some pulled out of the beam concrete (Figure 7.3).
- A few stair units were left hanging vertically from the starter bar reinforcement. Refer to Figure 7.4.
- The units that became detached from the structure came to rest at the bottom of the stairwell. Refer to Figure 7.5.
- Some bottom landings showed cracking consistent with a shear failure under axial load and flexure. Refer to Figure 8 of the *Site Examination & Materials Tests Report*.
- Some units showed significant cracking, bar yield and plastic rotation in the region between the landing and the sloping portion of the unit.
- Significant sliding on the lower support. Refer to Figure 7.6.
- At least some of the seismic joints are said to have been filled with construction debris, polystyrene packing, and/or mortar. This observation was also made after the 4th September 2010 earthquake. Refer to Figures 7.7, 7.8 and 7.9 (all taken after 22nd February 2011).

Further photographs can be found in the *Site Examination & Materials Tests Report*.

USAR engineers

Figure 7.2 : Fractured Top Starter Bars

Hyland

Figure 7.3 : Top Starters Pulled Out

USAR engineers

Figure 7.4 : Stair Flight Hanging from Top Starters (photograph taken from the top stair landing looking down the stair shaft).

USAR engineers

Figure 7.5 : Fallen Stair Flights at the Bottom of the Stairwell

USAR engineers

Figure 7.6 : Permanent Displacement at Bottom-Landing Seismic Joint

USAR engineers

Figure 7.7 : Mortar-Filled Seismic Joint

USAR engineers

Figure 7.8 : Debris/Mortar in Displaced Seismic Joint

USAR engineers

Figure 7.9 : Debris/Crushed Mortar below Sealant Backing-Rod

8 Evaluation/Analysis of the Collapse

8.1 Structure Condition/Capacity Prior to Collapse

The original design calculations were not made available for this investigation. The standard adopted or targeted for the design of this building is therefore not certain, but the date of the design would suggest that it was likely to have been carried out in accordance with NZS4203:1984.

In addition to prescribing the design lateral loads to be applied, and the way in which building deflections should be calculated, NZS4203 also defined specific requirements for separation of elements such as stairs. These specific requirements required that stairs be separated from the structure so that there would be no impact when the structure deformed to twice the calculated building deflections.

Calculations completed in accordance with NZS4203 as part of this investigation, suggest that the maximum inter-storey code drift is approximately 17 mm leading to a separation requirement for the stair seismic gap of 34 mm. This is not significantly different from the 30 mm seismic gap that was specified on the drawings.

However, there is evidence to suggest that at least some of the seismic gaps were compromised at the time of the 22nd February 2011 earthquake, having been grouted or accidently filled with construction debris or polystyrene. This is likely to have significantly reduced the clearance available.

Calculations completed as part of this investigation indicate that the stairs had sufficient strength to carry the self-weight of the units and the live load specified by NZS4203.

Inspection of the collapsed stair units indicated that the reinforcing steel and the concrete dimensions were generally in accordance with the construction drawings.

Testing of the concrete and reinforcing steel indicated that both were of the expected quality. Refer to the *Site Examination and Materials Test* report for the testing that was completed.

Cracking to the soffits and deformations of some stair units was observed after the 4th September 2010 earthquake. Although the seatings of the worst units were checked and confirmed as satisfactory at that time, recommendations in the Level 2 Rapid Assessment to carry out a more detailed inspection of the stairs do not appear to have been implemented. However, given the limited nature of cracking that occurred in September, it is considered that this is unlikely to have had a significant effect on the collapse that occurred on 22nd February 2011.

No further damage to stairs was reported after the Boxing Day 2010 earthquake, and none would have been expected - given the level of shaking that occurred.

Building occupants interviewed have stated that repairs to earthquake damage to floor coverings on the stairs in the period between the September 2010 and February 2011 earthquakes were underway.

8.2 Effects of Earthquake on Site and Structure

The analytical model of the structure that was described in Section 5.2.3 has also been subjected to the REHS record from the 22nd February 2011 earthquake using a time-history analysis to estimate the extent of the inter-storey drifts and the way in which they varied during this earthquake. Refer to Appendix A3.

The reports of damage to the building structural frame indicated that it suffered only minor-to-moderate damage during the earthquake. The time-history analyses were therefore completed using an elastic model of the building structure with increased flexibility of elements to allow for cracking.

Our analyses predicted the changes in inter-storey drift (relative horizontal displacement between adjacent levels) with time for Levels 13 to 14 for the 22nd February earthquake that are shown in Figure 8.1. The drifts between these two levels were the largest in the building for the analyses that were carried out. It can be seen that the maximum drift predicted for this earthquake was of the order of twice that for the 4th September 2010 earthquake (refer to Figure 5.5), although the number of cycles during the September event was greater.

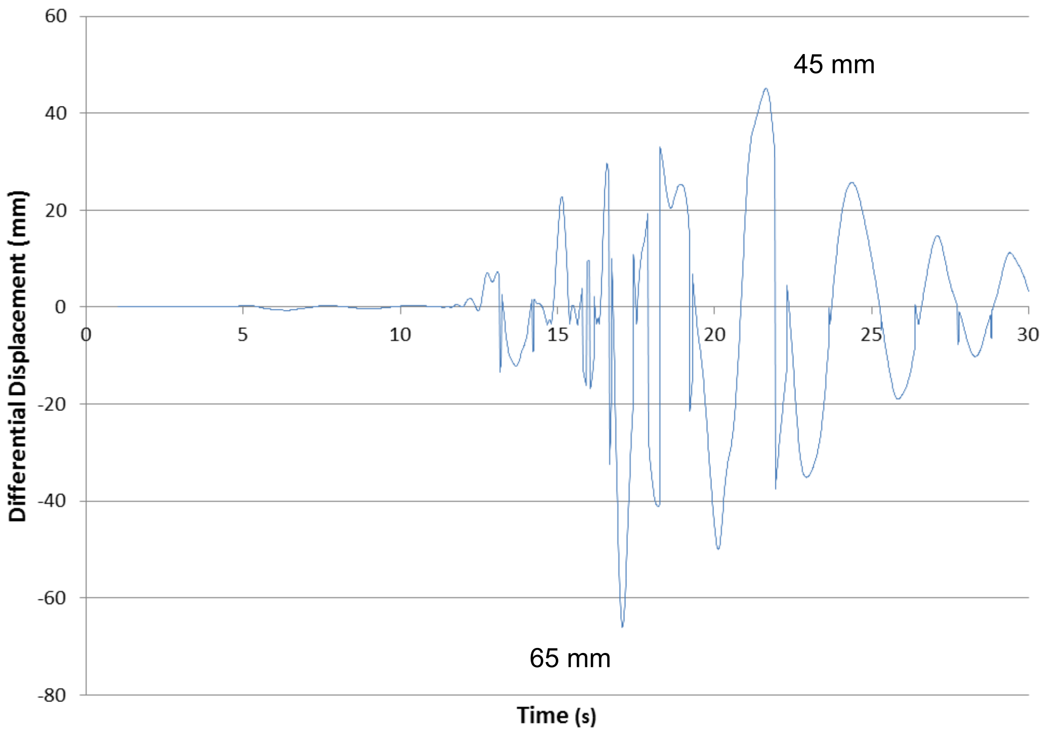

Figure 8.1 : Predicted Variation with Time of the Inter-Storey Drift between Levels 13 and 14 during the 22nd February 2011 Earthquake Obtained from Time-History Analyses Using the REHS Strong-Motion Record.

The level of drift predicted by these analyses (65 mm) is greater than we determined was required to be considered for design (34 mm) in accordance with NZS 4203 by a factor of nearly two. When compared with the clearances and seatings provided it also exceeded the seismic gap by a factor of two, but was less than the seating length that had been detailed (72 mm, assuming this had not been eroded by construction tolerances). The level of drift in this earthquake also exceeded the drift requirements of the current code, NZS1170.5, which have been estimated at 40 mm.

Displacements predicted by this type of analysis are dependent on the stiffness of the building analysed, but are not believed to be sensitive to the assumptions made regarding the extent of damage in the structure.

Separate models of the stairs have also been investigated for loading both along and across the stair. These inelastic models were subjected to increasing levels of inter-storey drift parallel to the stairs, with the following findings:

- For inter-storey drifts less than the clearance available in the seismic gap, the stair flights are able to move almost unrestrained, with only minimal induced actions due to friction between the toe of the bottom landing and its seat.
- Once the gap has closed, the stair flight becomes a strut between the connecting floors, and is placed into compression.
- The compression force causes the flight to bend downwards because of the kinked shape of the stair unit.
- A further drift of approximately 4 mm (i.e., beyond that required to close the gap) is sufficient to yield the reinforcing steel on the stair soffit below where the bottom landing meets the first step.
- Any additional drift is "locked in" as the reinforcing steel yields further. This leads to a shortening of the stair unit. The extent of locked-in shortening is the difference between the size of the original gap plus 4 mm, and the drift that occurs.
- On load (drift) reversal, the stair unit which is now shorter slides on the bottom support - but the seat is now less effective by the extent of the locked-in shortening.

Analyses have also been completed to investigate the effects of vertical acceleration and drift across the stair axis. Neither of these was shown to be significant, and therefore they are considered unlikely to be significant contributors to the collapse.

An investigation of the potential shear planes in the lower landing indicates a failure plane is possible at approximately 30 degrees from the horizontal and extending from the lower support point when the flight is subjected to axial load. This angle could be expected to become shallower when flexural effects are also included. The observed failure planes are at an inclination of between 15 and 20 degrees to the horizontal - which is consistent with the calculations.

The analyses outlined above and the assumptions made are described in further detail in Appendix A3.

Although some ground settlement may have occurred during the February earthquake, it is considered unlikely that this would have been sufficient to affect the stairs.

8.3 Consultant Evaluation of and Reasons for Collapse

From the information and data that has been gathered and the analyses that have been completed, our hypothesis as to why the stairs in this building collapsed is as follows:

- During the earthquake, the building inter-storey drifts were sufficient to cause the lower landings of (at least) some of the stair units to slide sufficiently on their supports for the available gap at the lower landing to close, and for these stair units to act as struts between floors.

  The required drift for strut action to occur is unknown because of uncertainty in knowing what the actual gaps were. It is known from observations, both after the September earthquake and of the remaining units above Level 15 after the 22nd February event, that the seismic gaps could have been filled with construction debris, mortar and/or polystyrene - thus significantly reducing the specified separation between stair and building structure.

  Optimistically, it can be assumed that the full 30 mm gap specified was available. Refer to Figures 8.2(a) and (b).
- When acting as struts, at least some of the units became overloaded and yielded in flexure at to the bottom step/riser. Analyses indicate that yield in this location will occur once inter-storey drifts of the order of 4 mm occur after the stairs become locked between floors (i.e., for drifts exceeding 30 + 4 = 34 mm). Refer to Figure 8.2(c).
- Yielding can also occur where the top step meets the top landing, but would not cause additional overall shortening of the unit.

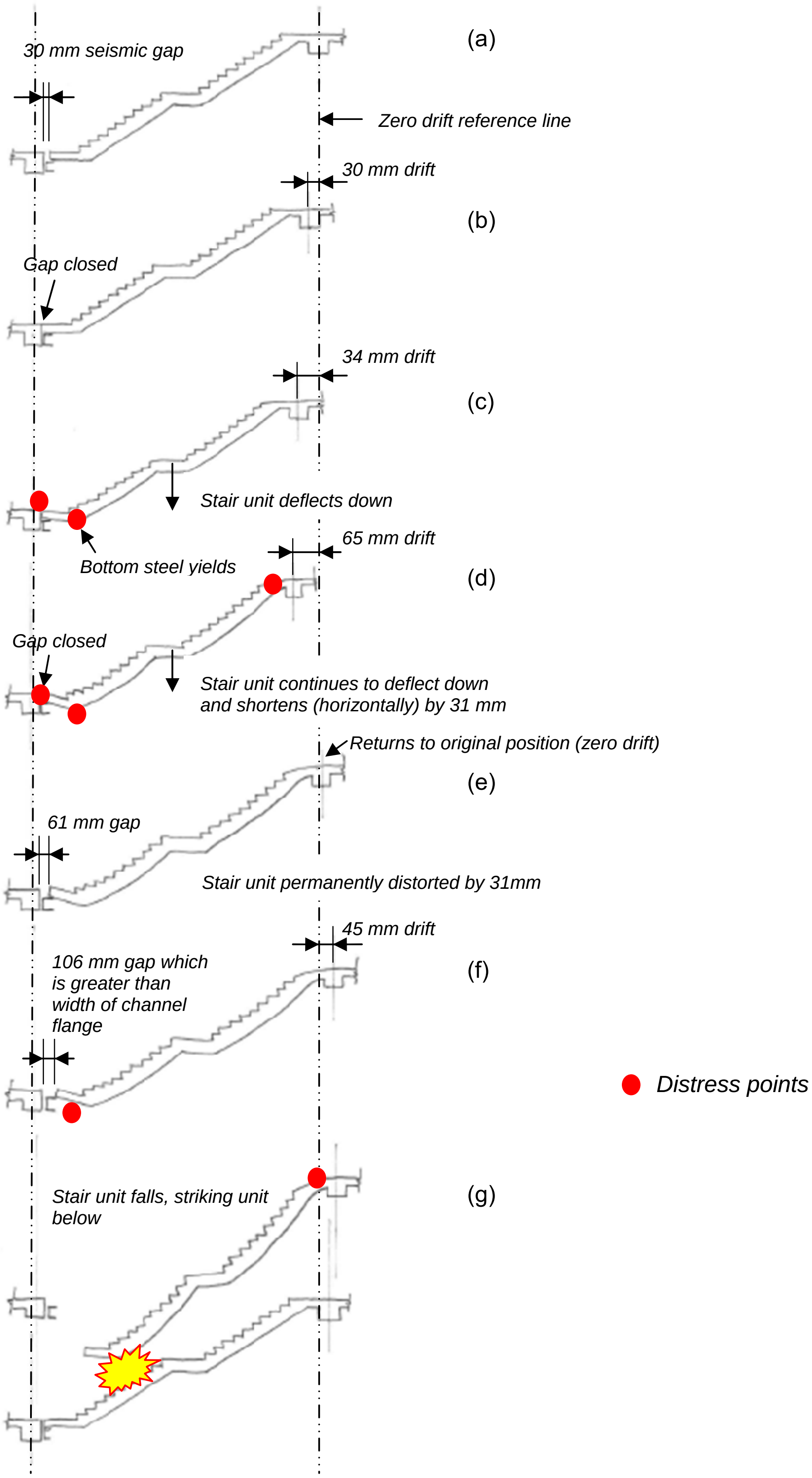

Figure 8.2 : Inferred Collapse Sequence

- Further inter-storey drift beyond 34 mm causes more flexural yielding, locking in further shortening of the unit.

  The analyses undertaken predict drifts for this building during the February earthquake of 65 mm between Levels 13 and 14. Thus, the extent of the shortening of the stair unit at this level could have been of the order of 65 – 34 = 31 mm. Refer to figure 8.2(d).

When the building returns to its original position (zero drift reference line shown in Figure 8.2) the gap at the lower landing is now 61 mm (4 mm elastic shortening having been recovered on unloading).

- The analyses predict a maximum drift in the opposite direction of 45 mm. When this drift is reached, the gap at the lower landing is 61 + 45 = 106 mm. This is greater than the total seat width of 102 mm available and the lower support of the stair unit is lost. Refer to Figure 8.2(f).
- The loss of only one unit would have been sufficient to cause progressive failure as each successive stair unit came into contact with the unit immediately below. Refer to Figure 8.2(g).
- Any filling of the seismic gap only increases the extent of the shortening of the stair unit and the likelihood that it will lose support on the reversing cycle.
- Collapse is also predicted if the drifts are first 45 mm in one direction followed by 65 mm in the other.

A diagonal shear failure across the bottom landing of a stair unit (seen in at least one of the units that did not collapse) is a possible outcome if that stair unit is loaded sufficiently in compression and has bent downwards.

Beca considers that the collapse of the stairs in the Forsyth Barr building on 22nd February 2011 occurred for the following reasons:

- The maximum inter-storey drift predicted by this investigation to have occurred in this earthquake exceeded the drifts required to be provided for by the Code used for the design of the building by about 80 %. This maximum drift also exceeded the current Code's requirement by about 60 %.
- The specified stair seismic gap, although considered to meet the requirements of the Code at the time of design, was insufficient with respect to today's Code.
- Any construction debris, mortar or polystyrene in the seismic gap would have significantly reduced the available gaps. However, collapse was still a possibility in the 22nd February 2011 earthquake even if the seismic gaps were clear.
- The width of the seating at the bottom landing of the stair units was insufficient to prevent a stair from falling when the drift was more than 54 mm.
- Once the support of any one stair unit was lost, there was the potential for a progressive failure if each unit collapsed sequentially on the unit below.

9 Conclusions

The way in which the stair collapsed has been hypothesised as follows:

- The sequence of the stairs collapsing has not been determined. It seems likely that the uppermost units collapsed first, possibly progressively spearing the units below.
- Interviews with occupants suggest that all the stair collapses occurred during the main shock over a short period of time. This is consistent with an upper-level stair failing and causing those below to fail progressively.
- It is likely that support at the bottom landing of one or more units was first lost, allowing the stair unit to pivot downwards about its upper end (which was cast into the building floor slab) until it struck and failed the unit below. In most cases, the cast-in reinforcing steel at the upper landing has then yielded and possibly snapped, presumably allowing the stair unit to fall down the building in a near vertical attitude. We have been advised that at least some of the units did not detach from their upper connections, and were left hanging in the stairwell.
- On any one stair unit, the lower seating support could have been lost for one of (or combination of) four reasons:
  - A stair unit has been compressed, resulting in bending downwards and yielding of the reinforcement - because the seismic gap was smaller than needed in the earthquake of 22nd February 2011. The resultant permanent shortening of the unit was sufficient for the lower landing to fall off the steel seat on the reversal of the inter-storey motion.
  - The lower stair landing failed in shear when the unit was subjected to compression after the seismic gap was closed.
  - The effective horizontal length of the stair unit was shortened when struck by the unit above after the unit above lost its seating and rotated downwards about its upper landing. The consequent V-shaped lower unit would have dragged its lower landing off its seat.
  - A free-falling stair unit simply "pole-axed" a still-intact unit below, causing it to fail catastrophically and fall.

The following conclusions have been reached regarding the reasons for the collapse of the stairs:

- The damage observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes is not considered to have altered significantly the stairs to make them more vulnerable in the 22nd February 2011 earthquake.
- The specified seismic gaps at the bottom landings met the requirements at the time of design.
- The actual seismic gaps at the bottom landings were too small for the earthquake shaking experienced on 22nd February 2011.
- The stair units were not designed to resist compression that would arise from the closing up of the seismic gap.
- The form and geometry of the stair's lower seating did not allow any latitude if the building inter-storey displacements (drifts) in an extreme event exceeded the width of the gap and seat provided.
- Any reduction in the specified seismic gap because of construction tolerances and/or construction debris, mortar or polystyrene within it would have reduced the level of inter-storey horizontal displacement (drift) required to fail the stair.
- Although the stair may have survived the February earthquake if the drifts required by the current Code had been allowed for, collapse might still have occurred if there had been any accidental filling of the increased seismic gap.

10 Discussion

10.1 Discussion of Key Issues as Necessary to Accompany Conclusions

The key issue that arises from the collapse of the stairs in this building is the allowance for lateral drift in details that have no resilience once the displacements/clearances available are exceeded.

In this particular case, the problem arises in relation to both the gap and the seating width available on the reverse cycle of motion.

While meeting the requirements of the Standard of the day, the stair support detail adopted in this particular case had several issues, including:

- Difficulty in providing a larger gap (i.e., bridging required).
- No allowance for displacements significantly larger than the Ultimate Limit State (ULS) drifts estimated by the design Standards/Code.
- Potential for the seismic gap/clearance to be easily compromised (i.e., to be filled accidently).
- No allowance for the potential effects of closing the gap and the actions that these might induce in the stair unit.

The stair's lower support detail basically provided no resilience (redundancy) in the earthquake of 22nd February 2011.

It is an issue that details such as this are used in a number of buildings around New Zealand.

10.2 Comments on Implications for Future Standards and Practice

The building displacements required to be accommodated to ensure the satisfactory performance of non-primary structural elements and interaction between structural and non-structural elements have increased with each successive Code revision. However, even the inter-storey seismic drifts that would be predicted for the ULS in the latest Standard, NZS1170.5, were exceeded during the 22nd February 2011 earthquake.

It is clear that there is sometimes a need to focus on the resilience of a particular part of a structure. An example is when there is no redundancy in the egress system - such as where is only one stair. Details, such as those used on the Forsyth Barr stairs, are not desirable or appropriate for such situations.

In NZS 4203:1984 there was a requirement to provide double the calculated gaps/clearances for stairs. This requirement was lost in later revisions of the Standard. Even though the drifts that are now calculated in accordance with the current Standard exceed the NZS4203:1984 values, there is perhaps a case, based around providing resilience, to have such a requirement reinstated in the Building Code. Such a provision would discourage the use of non-resilient details.

A common alternative detail that is used to support stair flights is shown in Figure 10.1. This detail does not have the disadvantages of the gap-and-ledge arrangement, and should be encouraged as an alternative in new construction.

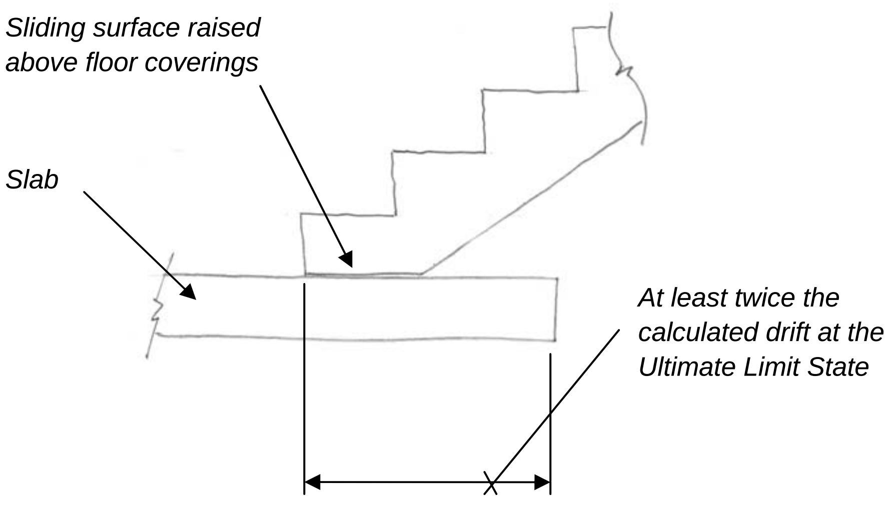

Figure 10.1 : Proposed Superior Base Stair Detail

11 Recommendations

Recommendations arising from this investigation are:

- Known alternatives to the seismic gap detail used in this building should be used on all new buildings, and for replacing the stairs in this building. These alternatives minimise significantly any likelihood of the stair collapsing because of insufficient allowance for inter-storey drift.
- DBH should issue an advisory note warning of the potential issues and lack of resilience with the gap-and-ledge stair detail for new and existing buildings.
- Consideration should be given to including a provision in the Building Code requiring clearances and seatings for stairs to be capable of sustaining at least twice the Ultimate Limit State inter-storey drift (in addition to allowances for construction tolerances).
- The concept that a specified seismic gap must not be compromised under any circumstances should be promoted.

12 References

Hyland Consultants Ltd, *Forsyth Barr Building Stairs: Site Examination and Materials Tests*, Prepared for Department of Building and Housing, 22th September 2011.

NZS 4203:1984 *Code of Practice for General Structural Design and Design Loadings for Buildings*, including amendments 1, 2 and 3, December 1984.

NZS 1170.5:2004 *Structural Design Actions – Part 5 : Earthquake Actions – New Zealand,* New Zealand Standard.