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INDEPENDENT ASSESSMENT ON EARTHQUAKE PERFORMANCE OF

INLAND REVENUE DEPARTMENT BUILDING – 224 CASHEL STREET

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

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Spencer Holmes Limited

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Introduction

This report has been commissioned by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes to review the performance of the Inland Revenue Building at 244 Cashel Street, Christchurch during the Canterbury earthquake sequence.

The report is based on documentation provided by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes. No internal inspection of the building has been undertaken.

Location of Building

The building is located at 224 Cashel Street, Christchurch City.

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

Geotechnical Site Assessment

Prior to construction of the building, a geotechnical investigation was undertaken on the site by Geotech Consulting Limited. The report is dated 10 August 2004 and was lodged with the Christchurch City Council on the 27 October 2005 with the building consent application for the piled foundations. At the time the investigation was undertaken the site was covered by existing buildings. A first stage investigation, consisting of four cone penetration tests (CPT) located in the street close to each corner of the site was undertaken by Site Investigation Limited in February 2004. The tests were carried out to refusal and reached depths of between 3m and 6.9m.

The above investigation was followed by SPT testing in direct push holes at each of the CPT locations. The SPT testing was taken to depths of 9.5m and was carried out by McMillan Drilling in May 2004. This phase of the investigation included a cable tool borehole drilled to 15m depth in Madras St.

A summary of the sub surface conditions based on the investigation is set out in table 4.3 of the geotechnical report. A copy of table 4.3 is included below.

Layer	*Depth*	*Average thickness*	*Description*
1	*0 - 1.0m (NW)* *0 – 2.7m (NE)*	*1m to 2.7m* *1.6 average*	*Silty sand to silt*
2	*1.0 – 3.2m (NW)* *2.7 – 5.9m (NE)*	*2.2 – 3.2m* *2.3m average*	*Sand*
3	*Top at 2.9 – 5.9* *Bottom 8.1 (BH1)*	*4.3m at BH1*	*Sandy gravel*
4	*Top at 8.1m (BH1)* *Base at 18m typical*		*Sand*
5	*18 – 23m*	*4 – 5m*	*Silt*
6	*23 – 35m*	*10m +*	*Gravel, 1st aquifer*

Table 4.3 ***Summary subsurface profile***

The water table was measured immediately after the test at 2.75m and 2.2m respectively in two of the CPT test holes.

The report identified that the site is underlain with shallow sand layers that are likely to liquefy with strong seismic shaking. The report also commented that there is also a small liquefaction hazard in the sands below the gravel (presumably referring to layer four). The liquefaction potential was assessed on the basis that the alpine fault earthquake with three peak ground accelerations of 0.12g, 0.20g and 0.30g.

Soil classifications were inferred from the CPT cone and friction ratios and the results of the liquefaction assessment were qualified due to the uncertainty of particle size distribution of the soils.

The geotechnical report identified that the liquefied layers may consolidate resulting in ground settlement. Ground settlement estimates were included in the report under table 5.4, the authors noting that the method of analysis was empirical and approximate only with perhaps a +/- 50 per cent margin to the numbers given.

It is understood that the building was intended to include a basement constructed over the building footprint at the time the initial investigation was undertaken.

The geotechnical report included the options of:

- over excavation of a confinable material below basement level and provision of relatively short length bored piles
- excavating the liquefiable sands and back filling with compact hard fill to support spread footings bearing onto gravel under the whole building
- Using piles. Piles were expected to be in the order of 12m –15m in depth founded in denser sands.

Ultimate pile capacities provided in the report are recorded in table 8.1, a copy of which is reproduced below.

Pile	***Depth (m) to pile tip below existing ground level***		
	6	***6.5***	*7*
150 Square	*40*	*55*	*95*
250 Square	*140*	*130*	*200*
450 dia	*290*	*310*	*320*
600 dia	*510*	*510*	*560*
900 dia	*1150*	*1,050*	*1,270*
1200 dia	*2,000*	*2,200*	*2,200*

Table 8.1 ***Ultimate Pile capacities (kN) Non Liquefiable***

The report recommended that:

> *these ultimate stresses given should be reduced by a bearing capacity reduction factor to give values of allowable ultimate bearing stresses to be used with fully factored ultimate limit state (ULS) loads in accordance with NZS 4203 1992. The capacity reduction factor of 0.5 should be used for all load combinations except those including earthquake over strength when a value of 0.8 is acceptable.*

Pile capacities were estimated assuming that the underside of the gravel is at 8m depth overlying the loosest sand as indicated by the lowest SPT result from boreholes. The report comments that:

> *Piles are clearly limited in that their likely capacities are much less than loads indicated for the building. For the 11,100kN internal column load needing 22,200kN ultimate load capacity, a group of ten 1.2m diameter piles would be needed. At sensible pile centres of 2.5m this would need a pile cap of about 6m square, considerably larger than a spread footing. However piles have a distinct advantage with carrying load along the perimeter of the building where shallow footings would be located on the edge of the reinforced fill and may need to be eccentrically loaded. Deep bored piles with greater capacity could also be used with pile tips at about 12m – 15m depth in the denser sands.*

The authors comment that the report had been prepared for the proposal as outlined in the introduction and that the report was prepared prior to the decision to delete the basement from the proposed building.

After demolition of the existing buildings on the site, a series of 11 further CPT tests were carried out on the site. The subsequent report commented that the results of one CPT that penetrated through the gravel layer identified that the underlying sand may be more susceptible to liquefaction than previously assessed. Subsequent review recommended that the capacity of the piles bearing on the gravels should be downgraded and that:

> *If all risks to liquefaction induced settlement damage to the building is to be avoided, deep piles of 12m – 14m could be used.*

Despite the extensive site investigation and careful evaluation of foundation options, significant differential settlement of the piled foundations occurred in the 22 February 2011 earthquake.

Geotech Consulting Limited reported to Colliers International on the performance of the building foundations after the 22 February 2011 earthquake. The report concludes that *The overall settlement pattern is consistent with that predicted. Differences such as different levels at all the corners, will be the result of the variable soil profile leading to different loading and pile restraint patterns, and to the nature and directivity of the shaking on the building itself. While the magnitude of settlement is perhaps greater than expected, a large part of that will be due to the larger than design earthquake loadings on the piles and the temporary softening effects on the soils around the piles.*

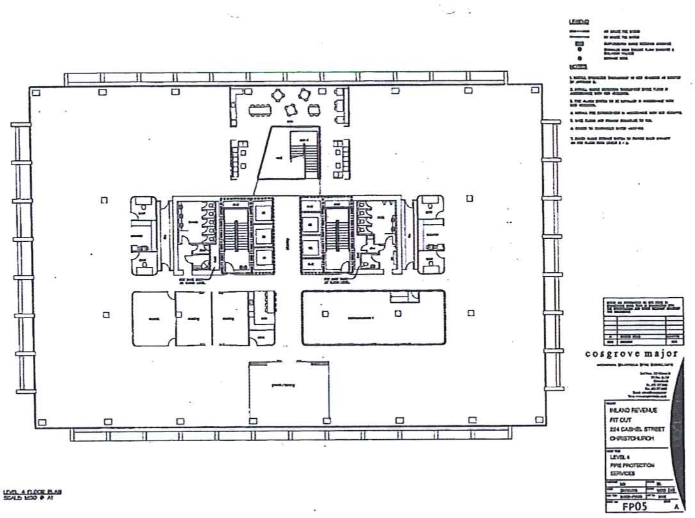
Description of Building

The building was built for Cashel Chambers Limited and construction was completed in December 2007.

The Inland Revenue Department Tower is an eight storey building incorporating approximately 2100sq metres of floor space. There are seven suspended floors in addition to the ground floor which is supported on grade.

A feature of the building is that the shear core and concrete frames are predominantly constructed of precast concrete elements that are joined together with insitu concrete joints.

The main floor plate surrounds the central lift, stair and services core. The building is clad with a curtain wall glazing system located behind precast concrete fins.



Typical Floor plan

Gravity System
The suspended floors are of 300mm thick pre-cast concrete hollow core floor units with a 90mm thick insitu concrete topping. The floors are supported on pre-cast concrete beams around the perimeter of the floor plate, the shear core walls around the perimeter of the shear core and by internal pre-cast concrete beams between the perimeter frame and the shear call walls. The pre-cast concrete beams are supported on pre-cast concrete columns. Structural steel roof framing supports a light weight steel roof. The plant deck is of reinforced concrete construction and sits directly above the lift core. The exterior reinforced concrete fin panels are connected to the perimeter gravity frame. The ground floor slab is a 100mm thick conventionally reinforced insitu concrete floor.

Seismic System
The primary lateral load resisting element is the central shear core which is assisted by a perimeter reinforced concrete frame.

In the longitudinal direction the shear walls are coupled shear walls of 550mm thickness with pre-cast elements being up to 10m in height. Generally the pre-cast units are interconnected with in-situ concrete. The coupling beams are reinforced with diagonal rods placed in ducts in the pre-cast panel and subsequently grouted The junction between pre-cast units at the coupling beams is a fully grouted 20mm wide joint. All horizontal joints between pre-cast elements are grouted.

In the transverse direction the walls are shear walls, 450mm to 650mm in thickness. These walls are constructed of hit and miss pre-cast and in-situ concrete with the pre-cast elements being up to 10m in height. All pre-cast panels are interconnected with vertical in-situ joints. Horizontal joints are grouted.

Secondary Elements
The main stair is located in the shear core and is constructed of pre-cast concrete stair flights with pre-cast landings and insitu toppings. Detailing of the stairs does not appear to make provision for inter-storey seismic movement.

Floor System
The floor system is a 300mm deep pre-cast Dycore flooring system with a 90mm thick in-situ concrete topping reinforced with 10mm diameter mild steel reinforcement at 300mm centres in the topping. Some additional topping reinforcement is provided at edges and other locations in the topping. The flooring system is provided with seatings at the pre-cast beams varying in width from 50mm to 75mm. The hollow core units have no seating at the shear core and are reliant on sloping dowels anchored into the shear core and into insitu concrete placed in the end of cores in the hollow core units. These dowels develop shear by shear friction under seismic loading.

Foundations
The gravity loads from the structure are supported on 900mm and 1200mm diameter bored concrete piles founded in dense sands at depths of up to 12m below street level. The shear core is supported off two interconnected 2.5 metre deep reinforced concrete rafts, in turn supported by the bored piles. The dimensions of each 2.5m deep raft foundation is 18.65m wide by approximately 10.70m long, a raft being located at the east and west ends of the main shear core. These rafts are spaced at 5700mm and 7000mm apart at the south and north end of the raft respectively. The eastern and western portions of the raft foundation are interconnected with 2500mm wide by 2500mm deep foundations beams. The foundation arrangements are recorded on Alan Reay Consultants drawing S1.03A7962.

Seismic loads from the shear core are transferred into the bored piles through the raft foundation.

Compliance

The Christchurch City Council issued a building consent for the project in four stages.

ABA10059660	Stage 1 – Piling
ABA12059660	Stage 2 - Foundations
ABA13059660	Stage 3 - Superstructure
ABA14059660	Stage 4 – Fit out

There is reference in the documents of a peer review having been undertaken by Opus Consultants, however we were not supplied with a copy of the review or documentation supporting the existence of such a review.

The compliance documentation appears to be in order and the Christchurch City Council issued a final Code Compliance Certificate on 16 October 2007.

Events Subsequent to 4th September 2010 Earthquake

The building appears to have suffered little damage from the 4 September 2010 and 26 December 2010 earthquakes but was significantly damaged by the 22 February 2011 earthquake.

The Canterbury Earthquake Royal Commission has been forwarded the following post earthquake reports prepared by Alan Reay Consultants:

Post earthquake site initial occupancy report 28 February 2011.

- This report was prepared following an inspection on 28 February 2011.

Post earthquake damage repair report 12 June 2011.

- This report primarily records the damage following the magnitude 6.3 earthquake on 22 February 2011 and aftershocks up until the date of inspection.

The author of the report recorded that Alan Reay Consultants understood that the main tenant, the Inland Revenue Department, had separately engaged a surveying company to undertake a full geometric survey of the building including levels and alignment and that CERA may have engaged another consultant to review the building. We have not been forwarded a copy of any such reports.

Structural Performance

Alan Reay Consultants undertook a limited level survey of the building on the ground floor and first level. Alan Reay Consultants comments on the results of the level survey are as follows:

> ***Ground Level*** *– the survey results indicated a wide variation in ground floor slab level. Some of this variation can be attributed to different as built floor levels to allow for floor finishes and a differential street level between two sides of the building. Some minor differential variation in floor slab levels was evident indicating localised settlement of the slab, however in general, the major finding was that the main structural core was in the order of 20mm to 90mm lower than the perimeter walls.*
>
> *Some areas of the floor were not accessible to survey.*
>
> ***Level 1*** *– Levels taken on level 1 indicate similar levels of settlement compared with the ground floor are evident. This confirms an overall settlement of the shear core relative to the perimeter of the structure. The slope on the floors at its maximum is 1:200 but in the general case it is closer to 1:600.*
>
> ***Level 7*** *– Levels taken on level 7 slab indicate general conformance with those levels taken on the ground and first levels. Whilst there is some inconsistent variation which could be due to factors such as differing floor finishes and construction tolerances the levels taken confirm the initial assumptions.*
>
> *The survey results therefore indicate that the shear core has settled relative to the perimeter frames or the perimeter frames have been pulled up and risen relative to the shear core. The cause, and more likely explanation, of the differential levels of the first option would be related to deep consolidation of the ground below the levels of the piles.*
>
> *The Geotech Consulting report prepared for the Inland Revenue Department and included in Appendix B agrees with the assumption that the shear core has settled relative to the other structure*
>
> The authors also identified various types and locations of structural damage:
>
> ***Cracks of Reinforced Concrete Perimeter Beams*** *– Cracks ranging in width between 1mm and 5mm have been identified, particularly in the corner beams, of the perimeter concrete frames. The cracks are typically a single crack which is evident in the topping concrete around the column and narrows to a fine crack at the underside of the beam.*

The cracks are most evident in corners of the building on all levels, however, similar cracks have been identified at other locations along the beam line column faces. This indicates that the cause is likely to be in part due to elongation of the perimeter frame as they have been cycled backwards and forward under seismic loads.

While these cracks do appear to be caused by flexural frame action there is a high likelihood that they are also partly due to differential settlements undergone by the structure. This is evidenced by the fact that none of these cracks were observed during the first visit to the building following the 22 February event.

The authors comments on the cracks having not been observed immediately following the earthquake supports a foundation settlement associated with site liquefaction.

Cracks in Hollowcore Units
Each floor contained several cracks in the hollow core unit between 0.5mm and 1mm. These cracks, in general are associated with the aforementioned cracking in the beam lines. The low tensile strength of the precast floor units in the lateral direction has allowed the unit to crack in the locations that the beams have elongated. These cracks have propagated through the insitu concrete floor topping and in some cases extend across the floor plate from the perimeter beam line to the shear core.

In four locations on the floor plate there is an insitu drag beam which is designed to draw lateral forces from the floor diaphragm into the end walls of the shear core. In some locations this is where the crack in the floor slab described above has occurred.

In one instance on the underside of level 1 suspended floor slab there is a crack in a hollowcore unit 50mm from its support that runs perpendicular to the span of the floor of approximately 0.5mm. The connection detail at this location of the unit to the beam includes insitu concrete in fills and reinforcing ties and therefore there should be no reduction in the load capacity of the floor.

The damage report would indicate that the hollowcore floor performance was satisfactory.

General cracking in spalling of concrete elements
Some minor cracking and spalling of concrete elements, particularly columns at beam column joints throughout each floor is observed. In nearly every instance this damage was of very minor nature and does not affect the structural performance of the building.

The report also includes comments on the ground floor slab damage and canopy collapse. The canopy that spans between the main office building and the adjacent pavilion car park building collapsed at one end. The report notes that:

The canopy that spans between the main office and the adjacent Pavilion Carpark building has collapsed at one end due to the differential movement between the buildings being greater than the Code design level.

In addition the report included recommendations for the repair of the cracks and further investigation. The further investigations related to:

- Breaking out concrete to inspect reinforcing at cracks in floor

- Observation of inside of hollowcore units by video camera
- Breakout of ground floor slab to allow visual inspection of shear core connection to foundation beams.

The authors comment that:

> *The damage recorded is either related to structural elements or architectural finishes and "fit-out items. Reviews of mechanical, electrical or fire services have not been undertaken as part of this report, however we understand relevant contractors have been engaged to ensure the continued operation of these items.*
>
> *In general the structural damage observed is of a minor nature and does not affect the primary lateral load resisting elements of the building. Various cracks to the floor slabs and perimeter reinforced concrete beams are associated with frame elongation where the perimeter reinforced concrete frames have been cycled under seismic loads. No significant failure of, or damage to, structural elements have been observed.*
>
> *The current assessment of the structural repairs required is that the majority of cracked elements will be left as they are in the short term except where they are exposed to weather or dust and debris in which case they will be covered with a sealant to prevent corrosion or filling or cracks. This will prevent any repairs further exaggerating any incidents of beam elongation given the current high levels of seismic activity that are expected to continue for some time. Final repairs can be considered, and undertaken, once it is apparent that seismic activity has receded to relatively normal levels.*
>
> *Some cracking to the floor slab was observed next to the drag beams perpendicular to the main core. Repairs to these areas are proposed to ensure full design strength is maintained in case of future similar events.*
>
> *In conclusion we consider that the building could be occupied subject to coordination of the immediately required repairs with other required works for occupation.*

The building performance in the Canterbury earthquake series has met the objectives of the New Zealand building code. The building suffered relatively minor damage and the glazing system and other secondary elements appear to have performed well.

While protection of the building structure is not an objective of the New Zealand Building Code, the building superstructure and the external glazing system demonstrates that many aspects of current design provide resilience to the damaging affects of major earthquakes. A review of the buildings performance identifies that a significant consequence of the 22 February 2011 earthquake is the apparent settlement of the bored piled foundations supporting the central shear core.

The period of the building is expected to be around 0.8 seconds in the transverse direction and 0.85 seconds in the longitudinal direction.

The February 2011 earthquake was particularly damaging to buildings with a period of 0.75 to 1.5 sec with GNS Science indicating that structures with a period between 0.75 and 1.8 seconds are likely to have experienced shaking more than 1.5 times more intense than the current loadings code prescribes.

It is therefore surprising that the coupling beams in the coupled shear walls and the lower levels of the main shear walls, which were designed for a ductility of 5, did not suffer significant inelastic deformation. Factors that may have affected the levels of inelastic deformation are.

- Use of high strength concrete
- Unsymmetrical inelastic behaviour
- Foundation performance

Issues Arising from Review

Analytical Considerations

The complexities of analysing structural systems incorporated into buildings is often over simplified in University research and in practice. In particular, the contribution of the pre-stressing in the pre-cast floor units and reinforcement in the topping to precast floor systems adjoining seismic resisting frames provides additional tension capacity to the beams, normally the top flange of the beam. While the code reliably restricts the elastic strength of the composite action, the contribution of the pre-stressing in the pre-cast floor units and the reinforcement in the topping to pre-cast floor systems is expected to increase as plastic hinge elongation occurs. The extent to which this occurs should be investigated further as it may significantly increase the capacity of the seismic resisting elements which could then exceed the capacity of the foundations.

Similarly, NZS 3101 restricts the extent of flange reinforcement that can be included under Cl. 11.3.1.3. While this requirement may ensure that a minimum level of strength is provided at flanged walls and may limit inelastic deformations, it seems inconsistent to design a shear wall element for a high level of ductility when elements of the wall may not yield under seismic action in one direction due to a significant reserve of flange strength. The reliability of foundations can only be assured when the capacity of the shear wall assembly can be reliably calculated and inelastic behaviour is fully predictable. It is recommended that code requirements used to assess the capacity of shear wall assemblies be reviewed with the objective of improving the reliable assessment of the capacity of the shear wall assemblies.

Capacity design procedures also need to consider material over-strength, in particular the effects of high concrete tensile strengths.

For reinforced concrete buildings to be resilient under severe earthquake induced loading, the performance of both lateral load resisting, gravity supporting and secondary elements need to be assessed at the maximum deformations likely to occur in the structure. This is an important part of a limit state assessment.

Effects of liquefaction

The susceptibility of the site to the effects of liquefaction is identified in the geotechnical site assessment. The building was founded on 900mm and 1200mm diameter bored concrete piles founded on the dense sands at depths of up to 12 metres below street level. The site investigation did establish the presence of a silt layer at a depth of 18 to 23 metres below the site.

The reported low level of damage to the concrete shear core indicates that deformations in the foundations may have effectively base isolated the super structure during the Canterbury earthquake series.

It is uncertain as to whether the deformations of the bored piles during the 22 February, 2011 earthquake were due to presence of the silt layers beneath the founding layer for the bored piles or to the loss of skin friction at the occurrence of liquefaction. Careful consideration needs to be given to the appropriateness of bored piles for sites prone to liquefaction.

During construction, the ends of the piles rest on the selected bearing strata. As construction progresses, the load is transferred to the bored piles with much of the load often being transferred to the sub-soils through skin friction. Further, the gravity loads on the bored piles are often significantly less than the loads that are imposed on the foundation system when developing the capacity of the superstructure under seismic loading. It is an unfortunate coincidence that the loading which imposes the maximum load on the piling system may also remove the contribution of skin friction through liquefaction. It is important that pile design under seismic loading should not include any allowance for skin fiction and that appropriate consideration is given to the effect of loss of skin friction during a seismic event.

Capacity design of foundations

As the building was designed to capacity design requirements, overloading of the foundations should not occur unless the strength of the super structure exceeds the assessed capacity or the reliable ultimate strength of the piled foundation is overestimated. The lack of inelastic deformation evident in the shear cores, which were likely subjected to seismic loading in excess of the loading code requirements, and certainly well in excess of the expected initiation of inelastic deformation for a structure designed for a ductility factor of 5, suggests that the structure may have a structural capacity in excess of the capacity of the foundations. Premature foundation failure may have resulted in reduced demand in the superstructure.

It is recommended that the appropriateness of the capacity reduction factor of 0.8 adopted in the derivation of ultimate design soil pressures for foundations be reviewed, particularly in respect of alluvial sites with materials prone to liquefaction. The expectation of improved soil performance under earthquake loading in areas prone to liquefaction may not be realised in practice.

High concrete tensile strength

We understand that the precast units were formed of flowable concrete with a 28 day strength as high as 90MPa. This concrete could be expected to have a tensile strength of 6MPa. This high tensile strength would significantly increase the initial stiffness and strength of the shear core and may have contributed to over-strength of the shear core which initiated a foundation failure. A review of actual strength of reinforced concrete elements and the adequacy of current code provisions for over-strength particularly the tensile strength of concrete is recommended.

A further aspect of the building performance which is worthy of further evaluation is the effect of relatively high tensile strength on the primary seismic resisting elements and the observation that inelastic deformation has occurred at a single crack in the seismic resisting elements rather than over a widely dispersed plastic end zone.

The concentration of inelastic deformation in ductile reinforced concrete frames at a single crack is a feature of the performance of reinforced concrete frames subjected to the Canterbury earthquakes. Such concentration of cracking has the potential to result in localised strain hardening and low cycle fatigue in the reinforcement. Consideration should be given to investigating this feature of the damage which is at variance with the wide zones of fine cracking that occurs in laboratory testing under progressively increasing multi cycle loading regimes. Factors that may be contributing are the speed of loading, the high tensile strength of the concrete in reinforced concrete buildings due to aging of the concrete and the significant reduction in stiffness that occurs once isolated cracking occurs.

This feature of the inelastic deformation of the Inland Revenue Building together with similar evidence in other buildings suggest that the loading regime adopted in laboratory testing may not be appropriate in developing design criteria for buildings which are likely to be subjected to earthquake ground motions similar to that experienced on the 22 February 2011 earthquake

Unsymmetrical inelastic behaviour of shear walls

The shear core to the building integrated the longitudinal and transverse lateral load resisting walls into a composite box structural form. The effect is that some wall elements have significant flange elements with reserve compression capacity and reinforcement at one end while the other end resists moments and shears from reinforcement and concrete within the thickness of the wall. As a result, some shear wall elements have an unsymmetrical response under earthquake with tension steel at the end of walls without flanged returns yielding while concrete tensile strength and/or the additional tension steel in the flange return suppressing yield in the steel at the flange end. The over strength of the flanges to the walls within the shear core may have contributed to over-strength of the core. The potentially unsymmetrical inelastic behaviour of shear wall elements with varying flange elements justifies further consideration and research, particularly the adoption of relatively high levels of ductility when symmetrical inelastic behaviour is prevented by the effects of flange over-strength. The unsymmetrical behaviour of L and T shaped shear wall elements in buildings subjected to the Chile Earthquake was a noticeable feature of damage to apartment buildings.

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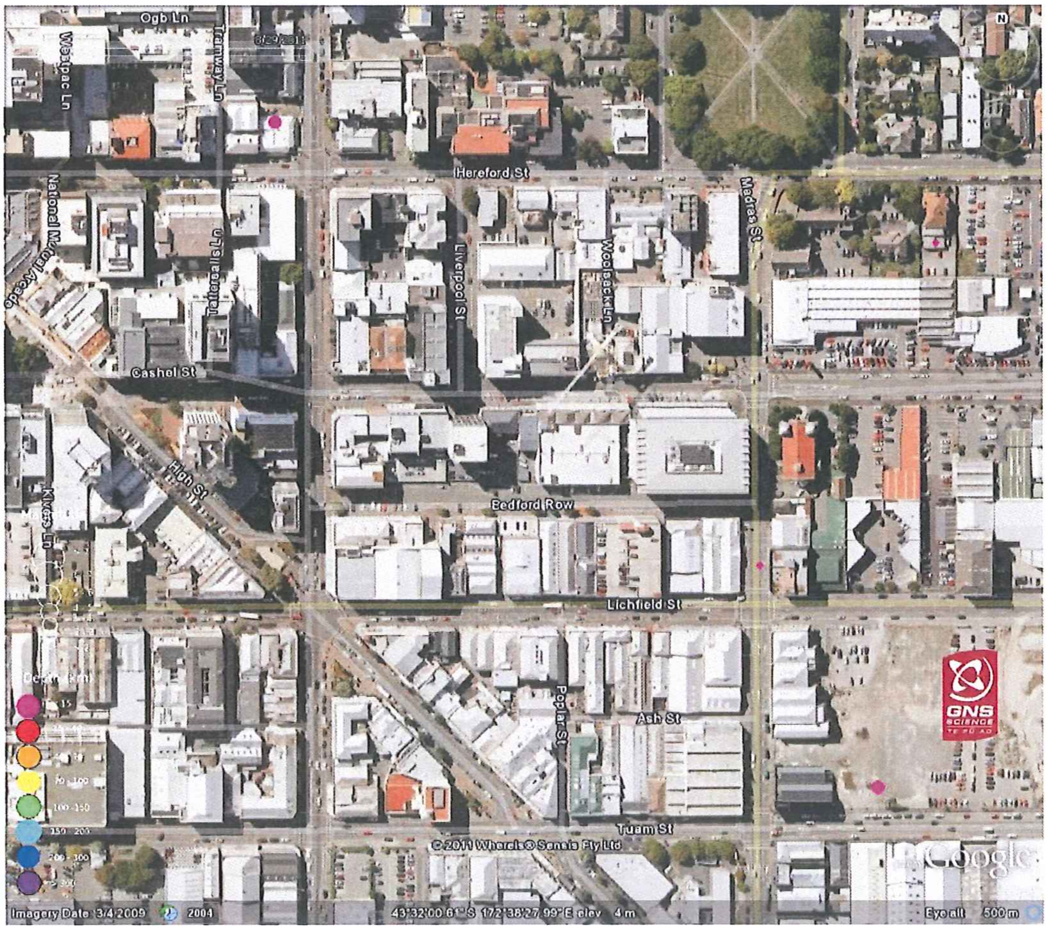
G/E110604-224 Cashel St – 22 Feb '12

APPENDIX 1

Site Plans


NORTH
Liverpool St
Woolsack Ln
Mw7.1 – 4th SEP 2010
No.224
Mw6.2 – 22nd FEB 2011
Bedford Row
Lichfield St


Ogb Ln
Tramway Ln
Westpac Ln
Hereford St
Madras St
Liverpool St
Woolsack Ln
Cashel St
High St
Bedford Row
Lichfield St
Poplar St
Ash St
Tuam St
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Google
GNS SCIENCE
Imagery Date: 3/4/2009
2004
Eye alt 500 m