Report for

Detailed Seismic Analysis
Forsyth Barr House
165 Stuart St
Dunedin

Job No: 13803
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Section A Executive Summary

Detailed seismic analysis of Forsyth Barr House, 65 Stuart St, Dunedin has shown that the buildings' structural elements have sufficient capacity to withstand the forces generated by 100% of the design earthquake loading for a building in its location.

The stairs in the building have sufficient flexural and shear strength to remain functional and allow egress from the building for the occupants in the event of an emergency.

Section B Building and Analysis

Our review is based on obtained copies of the original 1967 structural drawings.

The geotechnical conditions at the site have been taken as Type C (that is neither rock nor deep and soft) in accordance with NZS1170.5 New Zealand Earthquake Loadings Standard. The choice for using Type C soils is from experience of foundation construction in this area and bore logs for this building. The GNS geological map describes the underlying geology for the area as 'Basaltic rock from the 2nd main eruptive phase of the Dunedin Volcano'. For the site to qualify for Type B, rock there would need to be less than 3m of overlaying soil or weathered rock. On this site the fresh rock is approximately 15m below ground level.

The building is an eight storey two-way concrete moment frame with a concrete block wall penthouse located on the concrete slab roof. The structure is supported on nominal 15m long, 1.0m diameter concrete piles. The floors are cast in-situ concrete slabs. The columns and beams are all cast in-situ concrete.

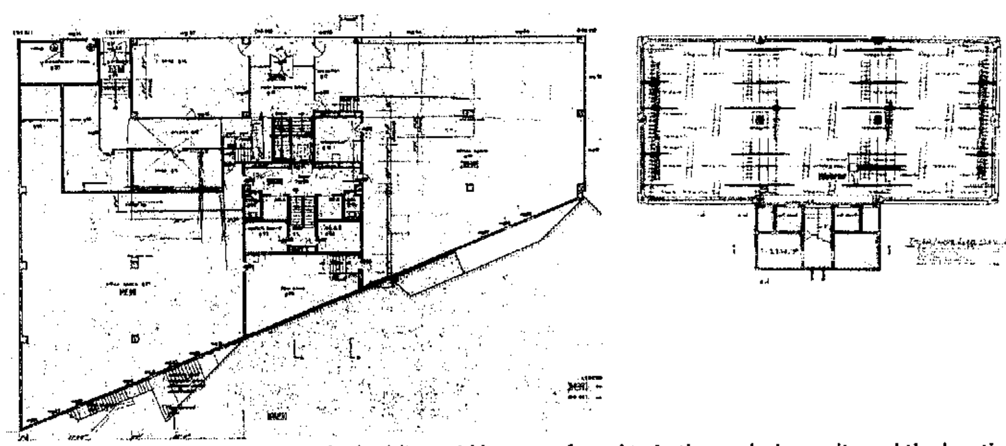


Figure 1 : 2nd floor plan showing the building grid layout referred to in the analysis results and the locations of the lift and stair service cores within the building and external stairs at the western end of the building.

The building has a large trapezium shape footprint up to the first floor level above which it changes to a smaller rectangular shape of the main tower block. The trapezium base of the building, although connected to the main tower does not provide any 'fixity' to the tower columns, with specially constructed joints at the intersection of the base floor and tower block columns and floor. The joint allows the base floors to rotate about the tower but not to move independent of the tower.

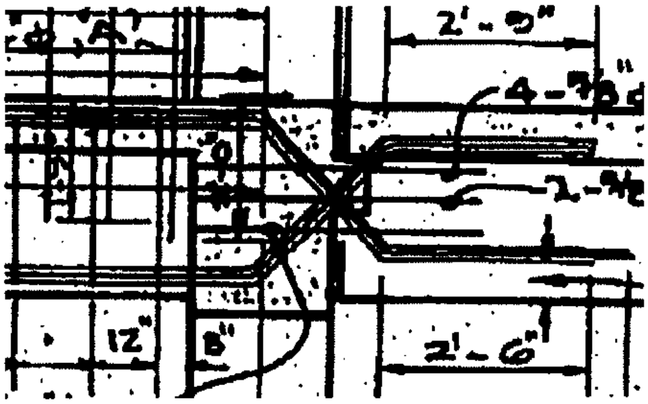
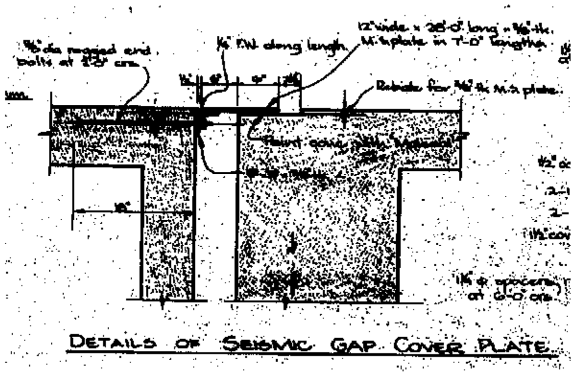


Figure 2: Showing the 'pin' joint at the connection of the main tower and the base floors.

The lift shaft and stair well are located on the exterior of the main tower on the south side of the building and is a separate structure from the main tower above the first floor where the building narrows. The connection between the two structures at the floor levels is a sliding steel plate over a gap of 150mm between the structures.



This gap means that the two structures can vibrate under seismic or wind loading completely independent of each other.

The structure was modelled using structural analysis software called ETABS and subjected to the loading regimes set out in AS/NZS1170, which is the current New Zealand design loadings Standard.

The building was analysed using the 'modal response spectrum method' as set out in NZS1170.5 Earthquake Actions, in accordance with the NZSEE document *"Assessment and Improvement of the Structural Performance of Buildings in Earthquakes"*

The moment frame tower block has a fundamental period of vibration of 2.2 seconds. The concrete walled stairwell which is much stiffer has a fundamental period of vibration of 0.2 seconds.

The building model was provided with material properties based on expected strength, which is higher than dependable or specified strength. This is the accepted procedure when analysing an existing building for resisting earthquake induced forces.
The expected strength of concrete was assumed to be 50% higher than the specified strength. The concrete stiffness used for design was increased by 30%. Reinforcing steel expected yield strengths were taken as being 10% higher than specified strengths as per standard practice.

The concrete member stiffness properties were modified to account for cracking. The effective stiffnesses for beams, internal columns and external columns were considered to be 50%, of gross sections.
The building was modelled as being 'fixed' at the pile caps, that is the piles were not modelled and the ground was considered to hold the columns rising from the pile caps in a 'fixed' position allowing neither translation nor rotation at their base.

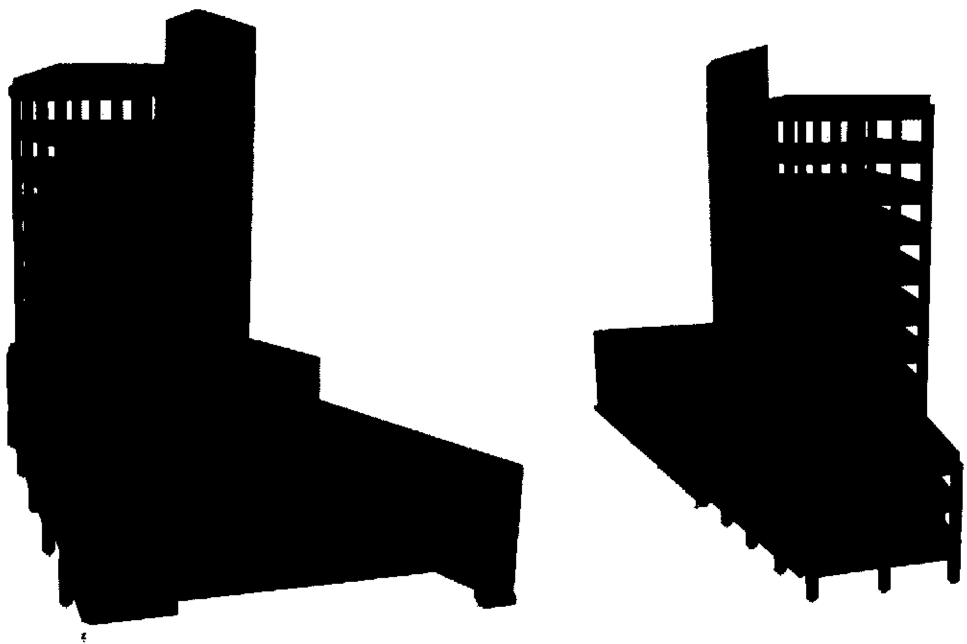


Figure 3: Model of building from ETABS structural analysis software. From upper Stuart St on the left and from above the Octagon on the right

Dunedin has a low earthquake risk when taken in a New Zealand context with approximately one third of the risk associated with a building in Wellington. All buildings in New Zealand are required to be designed to a 500 year return period earthquake, unless modified for buildings that may contain large crowds or particularly important or unimportant buildings. A five hundred year return period earthquake in Dunedin is significantly less severe than the 500 year return period

earthquake in more at risk parts of New Zealand. This is reflected in the earthquake induced forces that are required to be applied to buildings in Dunedin.
The minimum code requirements are to preserve life and to prevent collapse rather than to ensure further use of the building. It is possible that even if the structure remains standing, that there may be a large amount of damage and the structure may be on a lean and require demolition after the earthquake. Design according to current codes is to ensure "life-safety" rather than to protect the building for further use.

The choice of structural ductility, μ mentioned above has a direct and profound influence on the seismic loading that the building is subjected to for comparison with Building Code requirements. If the reinforcing in the building has been designed such that the building elements will fail in prescribed places, where building collapse is prevented under seismic loading, and the reinforcing at those places is detailed so as to hold the failed members together even though they are broken, then the structure is said to be 'ductile' If a structure is not ductile (ie $\mu = 1$) then it is must be subjected to the full Building Code seismic loading for a building in that location. If a building has a structure ductility of $\mu = 2$ the seismic forces are halved from the Code requirements for a building where $\mu = 1$. Structure ductility is usually in the range of $\mu = 1$ to $\mu = 6$.

Earthquake Loading Parameter	Value	Comments
Structure Period	2.2 s	Office Block, 8 Storey Moment Frame
	0.2 s	Stairwell, 10 Storey structural wall tower
Site Subsoil Class	C	> 3m depth subsoil
Hazard 'Z' Factor	0.13	Dunedin - NZS1170.5 Fig 3.4
Structure Ductility	1.3	Minimum required for %100NBS
Return Period Factor,	2	Normal Use
Near Fault Factor	1	> 20km from major fault

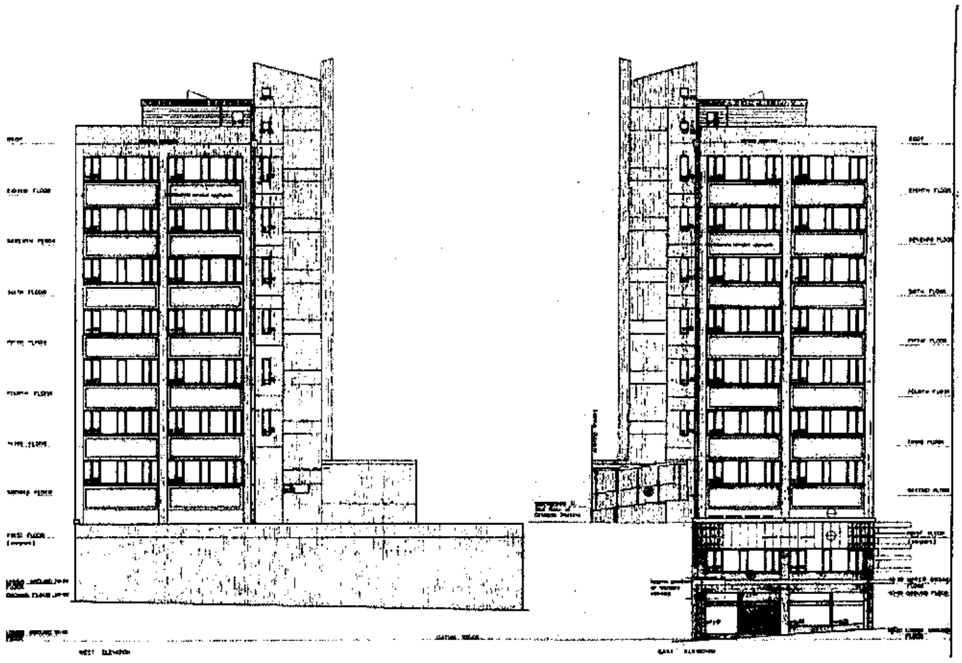


Figure 4: East and West Elevations of Forsyth Barr House from the Architectural drawings showing shear wall stairwell structure relative to the moment frame main office block.

Results

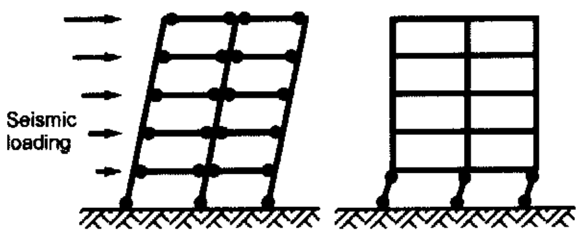
Display of the analysis results on the model indicates the most stressed representatives of repeated members throughout the building under the two earthquake loading directions. Capacity demand ratios have thus only been calculated for the most 'at risk' building elements not for every element in the structure.

The capacity to demand ratio (c/d ratio) of the various members throughout the building gives an indication of which members will fail before the others under a set loading, eg the 1170.5 design earthquake loading. A capacity/demand ratio greater than 1 means the member will not fail under that loading and less than 1 means that failure will occur. Members with a lower c/d ratio will fail before those with a higher c/d ratio.

1. It is evident from the earthquake demand / capacity ratios for the various elements of the building that a form of 'capacity design' has been carried out in the design of the building members. Frame buildings such as the office block of Forsyth Barr House will collapse when the strength of critical regions of the beams or columns is exceeded and 'hinges' form in the members. When certain combinations of hinges occur the frame forms a 'mechanism', can no longer resist lateral load and will collapse under further earthquake shaking. In the capacity design of structures design 'hinges' are allowed to form in places that will not

cause building collapse and the members are made stronger in the places where hinge formation would lead to collapse of the building. Where the hinges are allowed to form special measures are taken to ensure the members stay intact and hold up the building, allowing the occupants to escape.

In the diagram below the frame with the beam sidesway mechanism will not collapse with the formation of plastic hinges where they are indicated but the column sidesway mechanism will.



(a) Beam sidesway mechanism **(b) Column sidesway mechanism**

The capacity/demand ratio of the various members throughout the building gives an indication of which members will fail before the others under a set loading, eg the 1170.5 design earthquake loading. A capacity/demand ratio greater than 1 means the member will not fail under that loading and less than 1 means that failure will occur. Members with a lower c/d ratio will fail before those with a higher c/d ratio.

In Forsyth Barr House the beams have been designed such that they have a lower c/d ratio than the columns meaning that a beam sidesway mechanism will be likely to form rather than the column mechanism. This means that although members in the building may have failed, collapse should be avoided and the occupants able to escape.

The analysis shows that the beams do have sufficient strength to resist the current earthquake loading, but their c/d ratio is lower than the columns which all have a c/d ratio greater than 1.0 under elastic response loading leading, to the conclusion that a form of capacity design has been undertaken.

Reinforcement detailing in this building is well set out with close spacing of transverse reinforcing in high risk areas. From Table 1 below a structure ductility of $\mu = 1.3$ (1/0.78) is required for the office block moment frame to achieve 100% NBS.

Table 1: Building Element Capacity to Elastic Response Demand μ = 1 showing the hierarchy of failure leading to beam sidesway mechanism.

Element	Failure mode	Capacity	Demand G+0.3Q+EQx	Demand G+0.3Q+EQy	Cap/Demand
Moment frame beam	Flexure	1270 kNm	1368 kNm	1622 kNm	0.78
Moment frame beam	Shear	438 kN	322kN	483kN	0.91
Moment frame column	Flexure	1506 kNm	960kN	1264kN	1.19
Moment frame column	Shear	761 kN	521 kN		1.46
Moment frame column	Shear	744 kN		687 kN	1.57

Assessment of the beams reinforcement in accordance with *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* suggests a structure ductility μ = 4 for the building, however the difference in the capacity/demand ratio between the beams failing in flexure and failing in shear is insufficient to ensure that the beams will fail in flexure rather than shear (shear failure is brittle and sudden rather than the desirable ductile failure). The New Zealand Concrete Standard requires that design for shear is based on the shear forces that result from the flexural 'overstrength' of the member, which takes into account higher than specified strength of the steel and concrete, strain hardening of the steel and other factors that are unaccounted for in calculations. This is nominally taken as 25% more than the published material parameters. Thus the capacity/demand ratio of the beam failure in shear should be at least 25% greater than the capacity/demand ratio of the beam failure in flexure. From above it is 17%. We are confident that the reinforcing detailing required for a structure ductility of μ=1.3 is present allowing for all the moment frame members to achieve %100NBS.

2. Inter-story drift is the difference in lateral deflection between two adjacent stories of a building subjected to lateral loads.
The accurate estimation of inter-story drift ratio and its distribution along the height of the structure is critical for seismic performance evaluation purposes since the structural damage is directly related to the inter-story drift ratio.
The current provisions in NZS1170.5 limit inter-story drift to 2.5% of the storey height between any two adjacent floor levels.
The interstorey drifts in Forsyth Barr House under current Standard earthquake loading are around 0.5%; well within the 2.5% limit between any two adjacent floor levels from NZS1170.5.

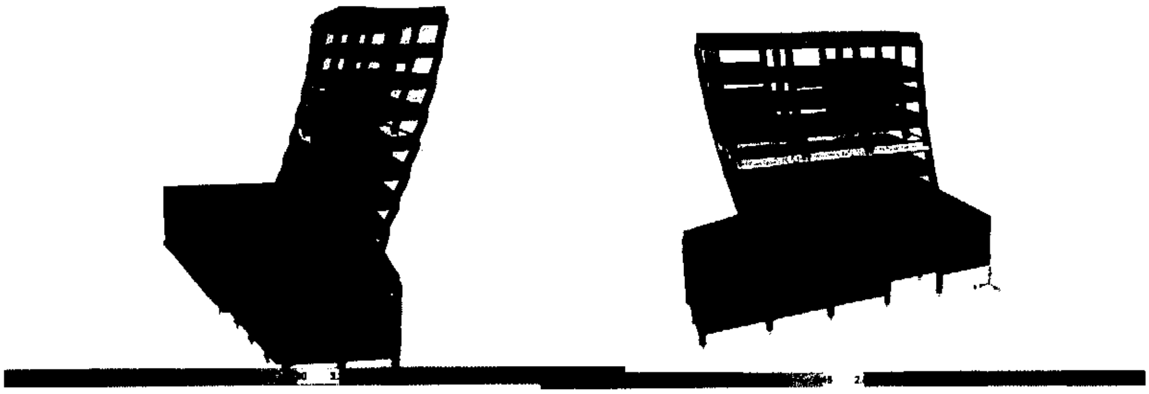


Figure 5; Showing exaggerated displacement of the office under north south earthquake loading on the left and east west earthquake loading on the right. It is evident from the displacement diagrams that the most stressed columns are on the second floor, which is verified by the second floor having the largest interstorey drift and the largest column moments and shears.

3. The wall of the stairwell that faces the moment frame office block is perforated with openings to allow access to the lifts and the stairwell. The wall is then effectively divided up into beams and columns of wall thickness. The beams tend to be short in length and deep in comparison. The flexural demands on such beams are usually low compared to the demand forces that they are subjected to but the demand shear forces tend to be high. This is the case in the wall of the stairwell with the openings. The worst shear capacity to demand ratio of 80% occurs on the 5th floor with a demand of 460kN compared to a section capacity of 372kN. However the location and proportion of these non-complying walls is such that their shear failure would not form a mechanism or compromise the overall stability of the building. Failure of this element is not 'likely to collapse causing injury or death to persons in the building' and thus does not lower our evaluation of the building achieving %100NBS.

In the stairwell all elements achieve 100%NBS except the spandrels beneath the lift entrances in the upper floors.

Table 2: Capacity/demand of stairwell elements; structure ductility μ=4

Element	Failure mode	Capacity	Demand G+0.3Q+EQx	Demand G+0.3Q+EQy	Cap/Demand
Stairwell wall spandrel	Shear	372kN	460 kN	204kN	0.8
Stairwell side wall pier	Flexure	5.7MNm	2.84MNm	2.14 MNm	2.0
Stairwell wall pier	Flexure	16.4MNm	2.6MNm	0.97MNm	6.3

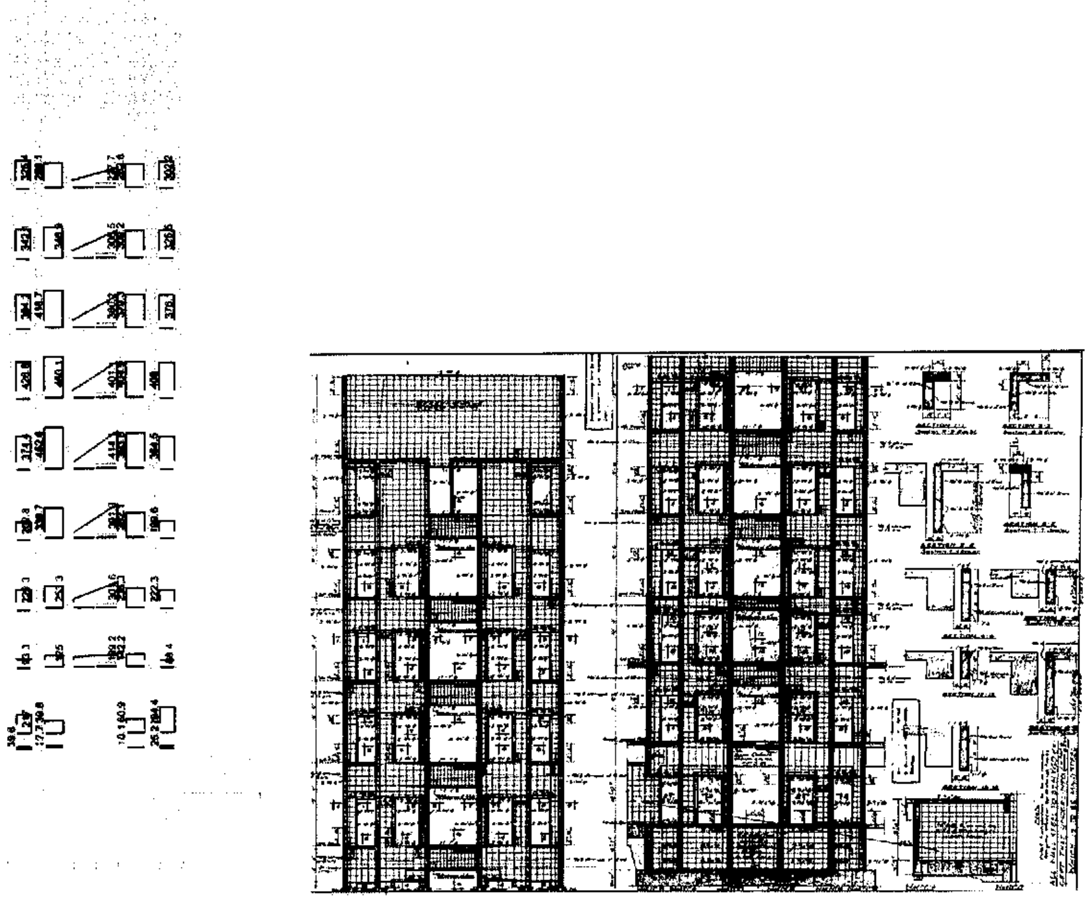


Figure 6: Wall of stairwell facing the office block. Shear forces from an east/west earthquake shown at left and reinforcing details at right

4. When two buildings are located close to each other, they may hit each other during strong earthquake shaking, causing damage; an effect called pounding. Theoretically, if the two buildings have the same characteristics, then under the same earthquake motions they should move together, in phase, without hitting, in the same way that windscreen wipers on a car move together.

However, due to different foundation conditions, different structural types and differing building heights, buildings seldom have the same characteristics.

Pounding will not occur during a design level earthquake, if the distance between the buildings is greater than the sum of the maximum displacements of each building alone without considering pounding.

The "seismic gap" between the tower block and the lift shaft is 150mm.

NZS 1170.5 requires that earthquake attack from two perpendicular directions be examined and this was carried out for the Forsyth Barr House analysis. The building suffers most displacement during north / south earthquake direction with displacements on the top floors being of the order of 140mm under the full elastic response spectrum ie with $\mu = 1$. Earthquake displacement from this direction, from shaking much larger than Building Code requirements, will induce the pounding damage described above with the independently vibrating stairwell. The much stiffer stairwell will only displace in the order of 10mm under the same loading regime thus pounding should not occur between the two structures.

5. A potential critical structural weakness that buildings may possess, that has been brought to the forefront by the Christchurch earthquake is the vulnerability of the stairs to collapse preventing egress from the building even though it may remain standing post-earthquake.

We have assessed the performance of the stairs as recommended by the Department of Building & Housing in accordance with the Report to the Royal Commission on Stairs and Access Ramps between Floors in Multi-storey Buildings.
NZS1170.5 has a design performance requirement that damage to non-structural systems necessary for building evacuation that renders them inoperative should be avoided.
At the level of interstorey drift that the stairwell experiences in the Maximum Credible Earthquake (MCE) cracking may occur in the stairs at the top level as the reinforcing leaves the elastic range but the stairs will remain intact and usable by building occupants leaving the building in the event of an emergency.

The building internal stairs in the service core are precast concrete construction. Reinforcing was left extended from the top and bottom of the stair flights, which was welded to exposed reinforcing on the landings and then cast in concrete. They are not attached at the sides to service core walls. The stairs have sufficient flexural and shear strength to resist the displacement of the interstorey drift at design earthquake levels.

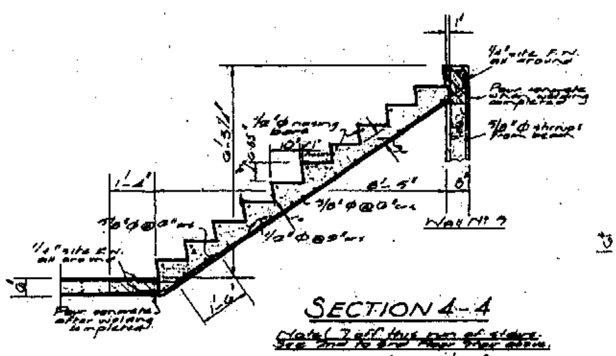


Figure 7: The stairs in the service core showing reinforcement on the left and the distribution of out of plane flexural forces that the stairs are subjected to in a north/south direction Maximum Credible Earthquake (MCE) scenario after which the stairs are to remain usable.

The stairs in this building have thus been found to meet the requirements of the Report and do not therefore represent a critical structural weakness in the building.

Section D Glossary

Critical Structural Weakness (CSW): a possible collapse hazard in a building. A significant vulnerability feature.

Plastic Hinge: used to describe the deformation of a section of a beam where plastic bending occurs. Plastic bending is the deformation of a material undergoing non-reversible changes of shape (yielding) in response to applied forces.

Ductility: a measure of a material's ability to undergo appreciable plastic deformation before fracture and thus to continue to carry load after yield and before fracture.

Bending moment: The product of a force and a lever arm. A force acting at a distance e.g. torque is a moment force as is a lever arm over a fulcrum

Shear: to deform or fracture as a result of excess torsion or transverse load

In-plane, Out-of-plane: bending or shearing motions which are respectively in the plane of the element and perpendicular to the plane.

Flexure: The act of bending

Mechanism: A system of elements whose connections are hinges and therefore cannot resist any loads applied to it.

Fundamental period of vibration: The elapsed time, in seconds, of a single cycle of oscillation. The inverse of frequency.

Out of Phase: The state where a structure in motion is not at the same frequency as the ground motion; or where equipment in a building is at a different frequency from the structure.

Overstrength: An allowance to be used when calculating the strength of structural members that takes into account factors that may contribute to strength such as higher than specified strengths of the steel and concrete, steel strain hardening, confinement of the concrete and additional reinforcement placed for construction otherwise unaccounted for in calculations.

Spectra: A plot indicating maximum earthquake response with respect to natural period or frequency of the structure or element.

Cast in situ: Concrete cast in its intended location.

Balustrade: An architectural barrier that is designed to provide building occupants with safety from falling.

%NBS: (Percentage New Building Score): Lesser of all the Capacity to Demand ratios of building elements that constitute Critical Structural Weaknesses for a building. The Capacity of a building element is evaluated in accordance with the appropriate current New Zealand Standard for the element material. The Demand is the earthquake induced force or displacement on the element ascertained from analysis in accordance with NZS1170.5 2004 the current New Zealand earthquake loadings Standard.

Section E Bibliography

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(6) Department of Building and Housing Advisory Group: *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury (revision 7): 2012.*

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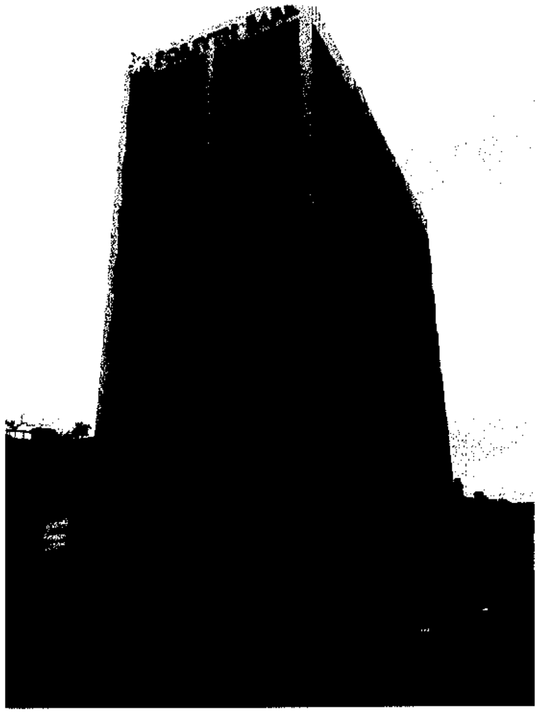
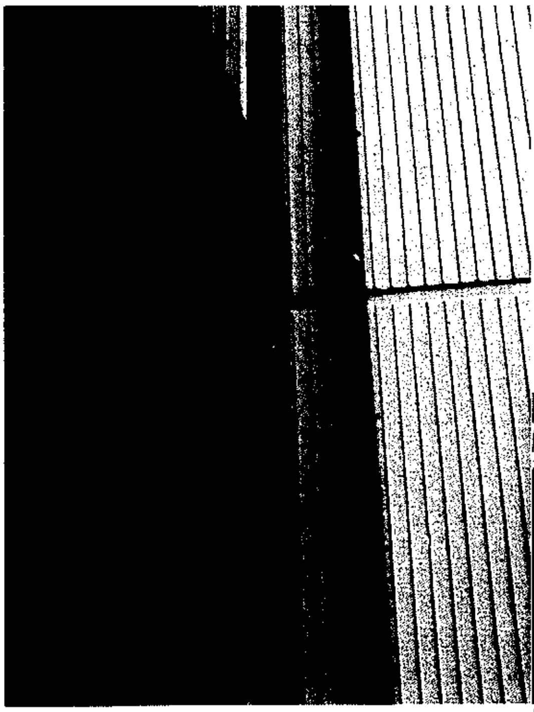
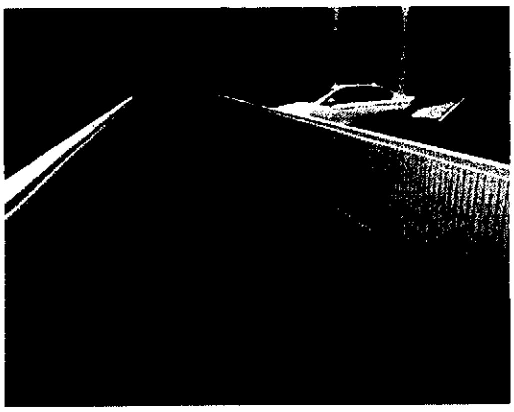
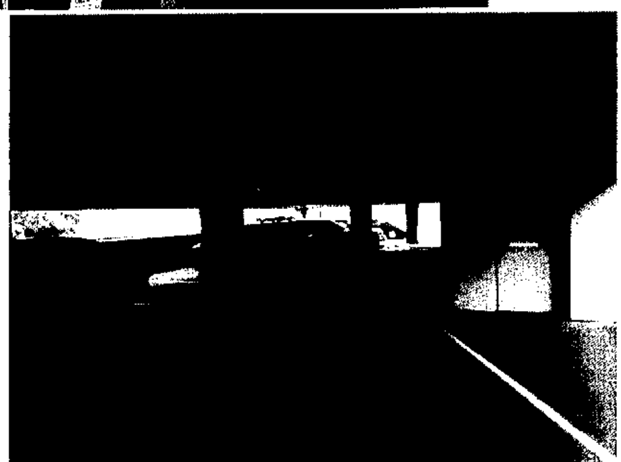
APPENDIX A Building Photographs



BARR HOUSE