

Report

314 The Terrace - Building Structure Condition & Detailed Seismic Assessment

Prepared for Victoria University of Wellington

Prepared by Beca Ltd (Beca)

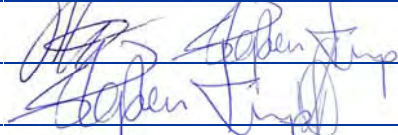
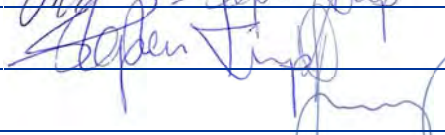

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Executive Summary

Beca has been commissioned by Victoria University of Wellington (VUW) to carry out a building condition assessment on the building at 314 The Terrace. As part of the commission Beca has also carried out a Detailed Seismic Assessment (DSA).

Building Structure

The building is an 11 storey reinforced concrete shear wall building with a single lightweight storey above the main concrete structure. It was originally designed in 1954 as self-contained state-housing apartments. Construction was completed in 1957. The floors alternate between reinforced concrete and timber construction up the height of the building.

Vertical load from self-weight and imposed loading is transferred through the reinforced concrete and timber floors to the shear walls located between the tenancies. The shear walls transfer this load to the pile caps and piles and then into the ground. The lateral load resisting system of the building is reinforced concrete shear walls in both directions. The concrete and timber floors act as structural diaphragms to transfer the load to the shear walls. The shear walls transfer the seismic load into the ground through the piles.

The east façade of the structure consists of insitu concrete columns with timber and glazed panels, and precast concrete plank panels between the columns and slab. The west façade (rear) of the building features walkways for access to the apartments. The condition of the east and west façades is poor. Extensive and severe corrosion of the reinforcement to the concrete slab edges and façade columns can be seen throughout the façade and falling pieces of concrete have been reported, further rotting timber and rusting steel columns are prevalent on the walkways.

The façade system will require either extensive rehabilitation or total replacement. It is noted that rehabilitation/repair may not be practical and risk of continual corrosion/deterioration of the exposed concrete elements will need to be considered as well as residual maintenance and repair obligations. Left unattended it is likely to result in safety issues with unsafe balustrade fixings and potential falling debris as currently exhibited. Continual deterioration and on-going maintenance could be minimised, but not necessarily eradicated, by adopting a certain wall type façade system that encloses the exposed concrete wall and slab edges.

The construction method of the piles is uncertain, but is believed to have consisted of boring the pile hole, placing the reinforcement and the dry concrete aggregate and finally pouring in a mixture of water, cement and sand (grout). We consider this method of construction is likely to lack reliability and may have compromised the integrity and strength of the piles. To ascertain the construction and condition of the pile invasive investigation and testing would be required. This is likely to be challenging due to difficulties accessing the piles. We note that there is currently no evidence that the foundation system is not satisfactorily supporting the building gravity loads.

The construction drawings indicate that the building is generally well detailed considering its age. Although it was built with plain round reinforcing bars, the lap lengths of the reinforcement in the longitudinal wall are approximately 90% of what is required today. The transverse shear walls are doubly reinforced, and the reinforcing bar spacing is reasonable. There are no ties between the two layers of reinforcing bars. The longitudinal spine wall is shown on the drawings to be primarily reinforced with diagonal bars. This allows the reinforcing to be continuous around the significant number of penetrations providing good load paths in tension and compression to carry the lateral seismic loads.

Detailed Seismic Assessment

Based on our assessment, we consider the superstructure of the building at 314 The Terrace to achieve less than 34%NBS and is therefore considered Earthquake Prone. This corresponds to a grade D building as defined by the NZSEE guidelines and exposes the occupants to a high risk relative to a new building. The building has been assessed using the NZSEE guidelines and assuming IL2. The assessment is limited by the rating given to the façade which has deteriorated significantly and to the point that its capacity has been significantly reduced.

If the façade is retrofitted the next most critical element achieves at least 50%NBS. This is governed by the tension yielding of the reinforcing bars in the longitudinal spine wall.

We are unable to comment on the capacity of the pile foundations with the information available. If, upon investigation and analysis, the foundations score lower than the superstructure and / or the facade, the %NBS of the building will need to be reduced.

Recommendations

We recommend the following work be carried out in order to achieve more certainty about the buildings seismic performance:

- Retrofit of the façade
 - Complete replacement of the existing facades with a curtain wall system or similar is likely to be the only practical solution
 - Concrete repairs to the existing exposed concrete slabs and wall edges by removing carbonated concrete, exposing and cleaning or replacing corroded reinforcement and applying new concrete repairs to reduce the possibility of falling concrete.
- Investigation into the piles.
 - Carry out piling investigations to a minimum of 2 piles by drilling through the full length of the pile to determine the concrete strength and integrity. The number of piles investigated may need to be increased depending on the results.
 - Break out the top of the piles to determine the reinforcement content and connection of the foundations to the superstructure.

In addition to the above, in order to achieve 80 to 100%NBS, the following work will be required:

- Investigation and confirmation of the diagonal reinforcing in the spine wall by breaking out sample sections of the wall.
- Work on the foundations including connection of piles to the walls. Dependant on results of investigations this may involve additional substructure and piling works.
- Increasing the capacity of the longitudinal wall with additional concrete walling (such as sprayed concrete thickening) may be required, this will be subject to the investigation works and further detailed analysis.

Alterations to the shear walls (i.e. removal of part or whole walls) are not considered practical and would likely require extensive structural retrofit works.

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Appendices

Appendix A

- Property Condition Photographs
- Building Structure Photographs

Appendix B

- Concept Design

1 Introduction

Beca Ltd (Beca) was appointed by Victoria University of Wellington to undertake a building condition audit and assist cost management assessment of 314 The Terrace, Wellington.

Our report is based on a non-invasive site inspection on 1 and 2 December 2014.

Our inspections were limited to sensory examinations of what we assessed to be typical parts of the building only where safe ready access existed at the time. Our inspections of the relevant aspects of the buildings as outlined above cannot guarantee that all possible facilities, defects, conditions and qualities are identified in this report. No underground services, hazardous material, geotechnical or subsurface investigations were undertaken.

This report is of defined scope and is for reliance by Victoria University of Wellington only, and only for this commission. Beca should be consulted where any question regarding the interpretation or completeness of our inspection and reporting arises.



Figure 1: 314 The Terrace Aerial Photo courtesy of Google Earth. Note it is erroneously labelled 320.

1.1 Regulatory Environment and Design Standards

Earthquake-Prone Buildings (EPBs) are defined in Section 122 of the *Building Act 2004* as buildings whose ultimate capacity will be exceeded in a moderate earthquake and would likely result in injury or death or damage to any other property. A moderate earthquake is defined as approximately one-third as strong as the earthquake shaking assumed in the design of a new building.

Using the *2006 NZSEE Guidelines* terminology, a building that achieves less than 34% of the New Building Standard (%NBS) is categorised as Earthquake-Prone. The *NZSEE Guidelines* also define a building achieving less than 67%NBS, as Earthquake-Risk. The *NZSEE Guidelines* recommend a minimum target strengthening level of 67%NBS.

It is considered impractical and unaffordable to design every building to withstand the largest earthquake imaginable. Consequently, with respect to the determination of design loads for natural hazards, the New Zealand Loading Standard adopts a probabilistic approach that takes into account the exposure hazard at a given location, along with factors such as building importance. Thus, the Loading Standard may be said to adopt a risk management approach in setting the loading levels that a given building is required to withstand.

For normal use buildings (e.g. offices, apartments), the “design” earthquake load is set at the 1 in 500-year return period earthquake event. This event has approximately a 10% probability of exceedence over the assumed 50 year life of a building.

1.2 Explanatory Statement

- This report has been prepared by Beca at the request of our Client and is exclusively for our Client’s use for the purpose for which it is intended in accordance with the agreed scope of work. Beca accepts no responsibility or liability to any third party for any loss or damage whatsoever arising out of the use of or reliance on this report by that party or any party other than our Client.
- The inspections of the building discussed in this report have been undertaken to assist in the structural assessment of the building structure for seismic loads only. This assessment does not consider gravity or wind loading or cover building services or fire safety systems, or the building finishes, glazing system or the weather tightness envelope.
- This assessment does not include an assessment of the building condition or repairs that may be required.
- No geotechnical, subsurface or slope stability assessments have been undertaken.
- Beca is not able to give any warranty or guarantee that all possible damage, defects, conditions or qualities have been identified. The work done by Beca and the advice given is therefore on a reasonable endeavours basis.
- Except to the extent that Beca expressly indicates in the report, no assessment has been made to determine whether or not the building complies with the building codes or other relevant codes, standards, guidelines, legislation, plans, etc.
- The assessment is based on the information available to Beca at the time of the assessment and assumes the construction drawings supplied are an accurate record of the building. Further information may affect the results and conclusion of this assessment. The information used to undertake the seismic assessment is listed in Appendix B.
- Beca has not considered any environmental matters and accepts no liability, whether in contract, tort, or otherwise for any environmental issues.
- The basis of Beca’s advice and our responsibility to our Client is set out above and in the terms of engagement with our Client.

2 Building Structure

2.1 Building Description

314 The Terrace, also known as the Gordon Wilson Apartments is an 11-storey structure with a one storey lightweight laundry above.

According to the commemorative plaque in the foyer of the building, it was opened in August 1957; hence construction preceding in 1956/57. The archive structural drawings show a date of 1954 for the design and documentation.

The building is currently unoccupied and was designed as residential accommodation consisting of two-storey apartments. Full height reinforced concrete walls act as party walls between the apartments. The floors at each level alternate between reinforced concrete and timber. The reinforced concrete walls split the tenancies, and the timber floors are an intermediate floor within each apartment.

A central reinforced concrete spine wall features in the longitudinal direction, with a number of penetrations made for accessing the apartments.

Vertical load from self-weight and imposed loading is transferred through the reinforced concrete and timber floors to the shear walls located between the tenancies. The shear walls transfer this load to the pile caps and piles and then into the underlying greywacke rock.

The building was constructed in the 1950s and from a review of the archive design drawings (not considered to be as-built drawings) it appears the building is founded on driven concrete octagonal piles, approximately 400mm in width. Pile lengths vary between 6.0m and 14.0m to follow the underlying rock profile and found the piles on clean greywacke strata. However, the February 1961 issue of *The Journal of the New Zealand Institute of Architects* discusses a different method of construction. It describes boring the pile hole, placing the reinforcement and the dry concrete aggregate and finally pouring in a mixture of water, cement and sand. Due to the dates of the information, we consider the latter method is more likely to have been used. There is no information available about the reinforcing content of the piles or the pile caps.

The lateral load resisting system of the building is reinforced concrete shear walls in both directions. The concrete and timber floors act as structural diaphragms to transfer the load to the shear walls. The walls transfer the seismic load into the ground through the piles. The shear walls in the transverse direction are 200mm thick and doubly reinforced. The spine wall in the longitudinal direction is primarily reinforced diagonally (single layer of reinforcing) between the penetrations which are staggered in elevation. This allows a direct load path around the penetrations in tension and compression, which is a positive attribute. There is also secondary vertical and horizontal reinforcing in the spine wall.

The façade to the building is constructed with a precast concrete column positioned centrally within the exterior of each apartment. The columns span vertically between the floors (both main concrete floors and intermediate timber floors within each apartment) and support timber and concrete external wall, panels and balconies.

The concrete slab edges and transverse walls are visible and exposed on the facades. The rear façade (west elevation) includes a partially covered walkway on every other floor providing access to the apartments.

To each end of the building there is a stairwell, consisting of cast insitu stairs and landings with glazed walls supported off the staircase structure. A separate lift tower is attached to the west elevation and connected with concrete floor slabs at each concrete floor location to the main structure.

According to the archive structural drawings, the stairwells appear to be founded on a strip footing and the lift structure is on a piled slab. The longitudinal wall appears to be placed on a ground beam along its full length, while the transverse walls are supported on ground beams and a pile cap arrangement with groups of piles at each end of the walls.

2.2 Building Structure Observations and Condition

A visual inspection of the exterior and interior was undertaken on 2 December 2014 by a Beca Structural Engineer.

The inspection was limited to areas freely accessible at the time of inspection and no intrusive investigations were undertaken.

2.2.1 Internal observations

A thorough walkthrough and visual inspection of the building was undertaken to gain an appreciation of the layout, structural arrangement and general condition of the structure.

Concrete walls between apartments and the main spine within the apartments running the length of the building appeared to be in a reasonable condition.

No cracking of the party walls or ceiling soffits was noted in the areas visually examined during the inspection.

The timber floors featured within the apartments appeared to be in reasonable condition and no signs of excessive deflection were noted. Potential for water ingress due to broken windows and gaps in the façade were noted adjacent to timber floor areas and therefore some deterioration of the timber could be expected.

2.2.2 Façade observations

Balustrades to the rear walkways feature steel handrails and in a number of locations throughout the building have corroded (refer image STRUC 01). Further the timber sills to the balustrades are rotting and deteriorating. Generally it appears the timber construction of these balustrades is in very poor condition and rotting in a number of locations (refer images STRUC 02, STRUC 03).

There are a number of vertical circular steel columns supporting the balustrade and handrail construction. Similar size rainwater pipes run vertically between steel column locations.

At a number of locations the top fixings of the steel column to the concrete slab edge have corroded. Significant concrete spalling to the slab edge at these locations can also be seen. In some locations, it appears that the top fixing is no longer positively fixed to the concrete and the column can be moved with modest pressure (refer image STRUC 04).

The slab edge soffit features a narrow drip detail. Concrete cover was often noted as missing at this location and reinforcing bars are exposed (refer image STRUC 05).

To the front elevation of the building (east face), the apartments feature a small inset balcony. The balustrade to the balconies is constructed with timber panels attached to the façade with steel handrails and infill mesh panels (refer image STRUC 06).

Fixings of the timber panels to the concrete slabs were generally in very poor condition and in a number of locations the fixings had completely corroded away. Restraint for the horizontal steel handrails had also been compromised in several locations (refer image STRUC 07).

Generally the facades are in very poor condition. Significant spalling of concrete and corrosion of the reinforcement can be seen in the majority of slab edge and wall/column junctions (refer image STRUC 08).

The concrete spalling is extensive in some places exposing corroded reinforcement to the base of the façade columns and offering very little residual strength against imposed wind and seismic forces. Further, the infill timber panels are connected to the concrete slab edges and columns and will be affected by the performance and fixing integrity of the concrete frame.

There have been reports of dislodged façade concrete falling to the ground around the building.

Within the stairwells, some of the steel balustrades/vertical spindles have corroded at the base where they are cast into the concrete stair flights.

2.2.3 Piling and substructure observations

As noted earlier in this report, there are differing methods of piling noted in the documentation obtained. The design drawings show octagonal driven reinforced concrete piles. A later Journal of the New Zealand Institute of Architects publication (February 1961 issue) describes the piling methodology as boring the pile hole, placing the reinforcement and the dry concrete aggregate and finally pouring a mixture of water, cement and sand. The piles are then left to set for several months.

As the Journal entry is dated later than the design drawings, we consider that the latter methodology was adopted for the piling system.

Construction of foundations for wharves and piers have, in the past used this technique of piling and anecdotally the results have varied in terms of quality, consistency and strength.

It is noted that the seismic performance of the building relies on the piling and foundation system to transfer the lateral and vertical loads into the underlying greywacke rock. Hence the condition, arrangement and adequacy of the foundations are key to determining the performance of the building.

3 Detailed Seismic Assessment

3.1 Assessment Methodology

The building has been assessed in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines, 2006 including corrigenda 1 and 2.

The following standards were also used in the assessment:

- NZS3101:2006 *'Concrete Structures Standard'*
- NZS1170.5:2004 *'Structural Design Actions' Part 5: Earthquake actions – New Zealand*

An inspection of the building was carried out on 2 December 2014. The inspection was visual only and did not involve any intrusive investigation.

The assessment was based on the drawing set by the Ministry of Works named "Multi-Storey Flats, Wellington" numbered GA6094 sheet 1 to 62. It should be noted that drawing 38 was not available and some assumptions were made about the reinforcing details that are expected to have been detailed on this. For the purpose of this assessment the structural drawings are assumed to be representative of what was constructed on site.

An outline of the steps used to assess this building is as follows:

- Calculate weight of the building
- Calculate site specific seismic demand spectra
- Carry out rocking wall analysis on transverse walls
- Create model to analyse longitudinal wall
- Carry out hand calculation of shear capacity of transverse and longitudinal walls
- Carry out hand calculations on seismic capacity of façade
- Independent verification of results by another Beca engineer.

3.2 Site Characteristics

The site is located on a hillside with varying depths of old slip and stream deposits over rock. Depth to rock in the centre of the building is approximately 15m and at the ends of the building it is approximately 6m. At the rear of the building the hillside was excavated and stabilised with a crib wall. The site subsoil class is a combination of B and C. The subsoil class has been conservatively assumed as C for the purpose of this assessment.

3.3 Building Description

3.3.1 Structural System

The building is an 11-storey reinforced concrete shear wall building with a lightweight storey above the main concrete structure. It was originally designed in 1954 as self-contained state-housing flats. Construction was completed in 1957. The floors alternate between reinforced concrete and timber construction up the height of the building.

A typical floor plan is shown in Figure 2. The lower plan shows the reinforced concrete floors, and the upper plan shows the timber intermediate floors. Figure 3 shows the north and east elevations of the building.

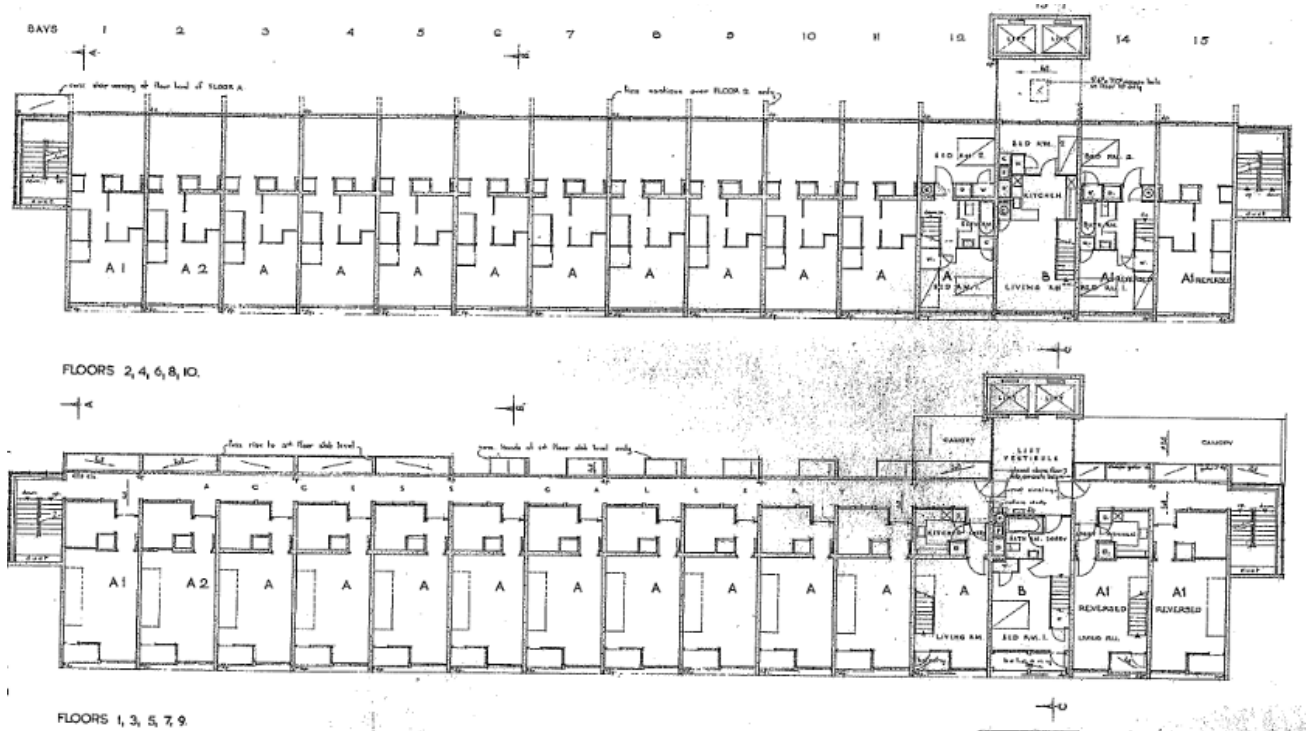


Figure 2: General Floor Plan

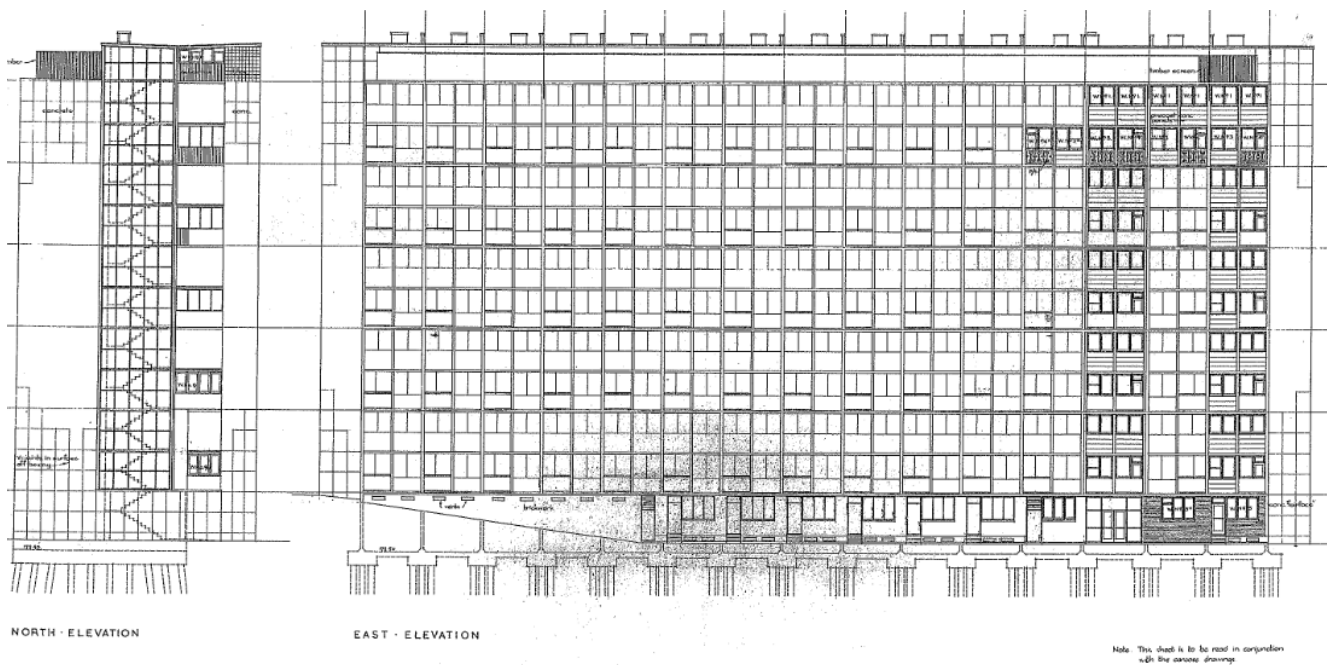


Figure 3: North and East Elevations

3.3.1.1 Vertical and lateral load resisting system

Vertical load from self-weight and imposed loading is transferred through the reinforced concrete and timber floors to the shear walls located between the tenancies. The shear walls transfer this load to the pile caps and piles and then into the ground.

The lateral load resisting system of the building is reinforced concrete shear walls in both directions. The concrete and timber floors act as structural diaphragms to transfer the load to the shear walls. The shear walls transfer the seismic load into the ground through the piles. The shear walls in the transverse direction are 200mm thick and doubly reinforced. The spine wall in the longitudinal direction is primarily reinforced diagonally (single layer of reinforcing) between the penetrations. This allows a direct load path around the penetrations. There is also secondary vertical and horizontal reinforcing in the spine wall.

3.3.1.2 Foundations

According to the archive design drawings, the building is founded on reinforced concrete piles. However, there are two contradicting pieces of information pertaining to their construction. The structural design drawings (dated 1954) indicate octagonal reinforced concrete driven piles, but the February 1961 issue of *The Journal of the New Zealand Institute of Architects* discusses a different method of construction. It describes boring the pile hole, placing the reinforcement and the dry concrete aggregate and finally pouring in a mixture of water, cement and sand (grout). Due to the dates of the information, we believe the latter method is more likely to have been used. There is no information available about the reinforcing content of the piles or the pile caps. No "as-built" drawings have been found for this building.

We are able to surmise that there was no requirement for tension reinforcing in the piles in the original design. Due to there being no reinforcing shown on the drawings, we believe tension reinforcement was not a critical part of the original design. A simple calculation approximating the design loads at the time of design confirms that there was no net tension demand on the piles under this lateral load.

3.3.1.3 Stair system

The stairs are located at the north and south ends of the structure external to the main floor plan. They are cast in-situ reinforced concrete stairs with walls encasing the staircase. A typical plan and elevation of the stairs is shown in Figure 4 below.

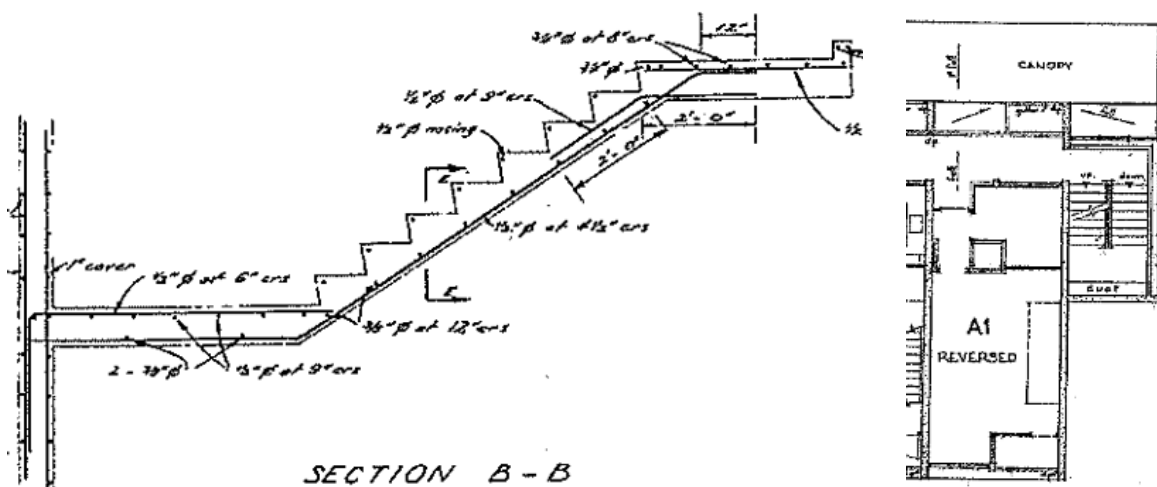


Figure 4: Typical Stair Layout

The stairs are fixed at both ends and therefore under seismic loading will move together. The surrounding walls are likely to provide sufficient stiffness that the stairs are unlikely to attract unwanted seismic load. This

means the stairs are unlikely to be damaged significantly enough in an earthquake that they become unusable or a life-safety issue, and therefore do not believe the stairs limit the score the building achieves.

3.3.1.4 Façade

The façade consists mainly of windows and precast reinforced concrete planks as cladding. These are supported by precast reinforced concrete posts that span between the floors. As discussed in the condition assessment, the façade has significantly deteriorated since the building was constructed. Figure 5 shows a typical area of deterioration of the precast post connection to the slab.



Figure 5: Façade deterioration

3.3.2 Building Design

The building was designed in 1954 by the Ministry of Works. The design date indicates it was most likely designed to NZSS 95:1939. The understanding of seismic engineering has vastly improved since the building was designed and the loading demand has increased significantly. Therefore, when a building of this age is assessed against the current code it starts at a significant disadvantage because it was designed to lesser loads.

The construction drawings indicate that the building is generally well detailed considering its age. Although it was built with plain round reinforcing bars, the lap length in the longitudinal wall is approximately 90% of what is required today. The transverse shear walls are doubly reinforced, and the reinforcing bar spacing is reasonable. There are no ties between the two layers of reinforcing bars. The longitudinal spine wall is primarily reinforced with diagonal bars. This allows the reinforcing to be continuous around the significant number of penetrations. The level of detailing is sufficient to allow a structural performance factor of 0.7 to be used.

4 Results of Seismic Assessment

Based on our assessment, we consider the superstructure of the building at 314 The Terrace to achieve less than 34%NBS and is therefore Earthquake Prone. This corresponds to a grade D building as defined by the NZSEE guidelines and exposes the occupants to a high risk relative to a new building. The building has been assessed using the NZSEE guidelines and assuming IL2 applies. The assessment is limited by the condition of the façade which has deteriorated significantly enough that parts of it cannot be relied upon in the event of a significant earthquake.

If the façade is retrofitted the next most critical element achieves at least 50%NBS. This is governed by the tension yielding of the reinforcing bars in the longitudinal spine wall. We are unable to comment on the capacity of the pile foundations with the information available. If, upon investigation and analysis, the foundations score lower than the superstructure, the %NBS of the building will need to be reduced.

Table 1: Relative Earthquake Risk

| Building Grade | Percentage of New Building Strength (%NBS) | Approx. Risk Relative to a New Building | Risk Description |
|----------------|--|---|--------------------|
| A+ | >100 | <1 | low risk |
| A | 80 to 100 | 1 to 2 times | low risk |
| B | 67 to 80 | 2 to 5 times | low or medium risk |
| C | 33 to 67 | 5 to 10 times | medium risk |
| D | 20 to 33 | 10 to 25 times | high risk |
| E | <20 | more than 25 times | very high risk |

4.1.1 Building Behaviour

The lateral load resisting structure in the longitudinal direction is the spine wall. The spine wall is governed by the tension capacity of the diagonally reinforced wall sections. The diagonal reinforcing and concrete struts act in direct tension and compression. Therefore, the reinforcing bars are able to yield and continue to displace and dissipate energy allowing a ductile mechanism to form. For this assessment we have used a ductility of $\mu=2$ as allowed in the NZSEE guidelines. The building achieves at least 50%NBS in the longitudinal direction, limited by the tension capacity of the reinforcing in the spine wall.

In the transverse direction the walls have been assessed as 'rocking walls'. The construction method of the piles and the likelihood that they were not designed to resist tension loading makes it prudent to consider them as having no tension capacity. Therefore, the walls are unable to achieve their full moment capacity because the piles are unable to hold the walls to the foundations. Once the inertia of the walls is overcome the walls will begin rocking on their foundations, which allows energy to be dissipated. Using this approach the building achieves at least 80%NBS in the transverse direction.

4.1.2 Foundations

Due to the uncertainty of the reinforcement and quality of concrete and pile integrity, it is difficult to identify the overall seismic rating of the piles, however a lower score for the piling system would reduce the overall building %NBS accordingly. As previously discussed, testing of the piles could confirm the pile type and integrity and assist with determining the overall %NBS.

4.1.3 Façade

As outlined in the condition assessment the façade has deteriorated significantly since the building was constructed. Therefore, any calculations on the capacity of the façade should not be relied upon as the deteriorated elements are unlikely to perform as the ‘as new’ calculations based on the drawings. It is not feasible to attempt to quantify the current capacity of the façade elements because such significant degradation makes it very difficult to estimate what is remaining from the original cross-section of the members. Engineering calculations on the “as-new” façade (based on the design drawings) indicates it achieves 100%NBS. This is based on the out-of-plane capacity of the posts and precast elements, and their ability to stay housed within the main structure.

The seismic capacity of the façade elements has been significantly reduced below “as-new” by the deterioration that has occurred. The rating of the façade is assessed to be less than 34%NBS and limits the rating of the building as a whole.

4.1.4 Ultimate Limit State

This assessment is based on the assumption that the building is deemed to be an Importance Level 2 structure, as defined by AS/NZS 1170.0:2002. Importance Level 2 correlates to a “normal” building, which is typical for most new buildings. Importance Level 2 structures, designed to today’s codes, are required to provide a high margin against collapse from an earthquake with a return period of 500 years. A new building designed to this level may be badly damaged after a major earthquake, perhaps beyond repair, but it should provide a high margin of safety for its occupants and allow them to exit the building safely after an earthquake.

In comparison to current code requirements, the main structure of the building at 314 The Terrace should achieve at least 50% of New Building Standard (%NBS) designed to Importance Level 2. The failure mechanism of the main structure of the building is likely to be tension yielding of the reinforcing in the longitudinal spine wall. This will potentially cause damage to the building to the point where it may no longer be economical to repair.

Even if the building is able to be re-occupied soon after a large earthquake, services within the building, such as water, power and lifts may be damaged.

4.1.5 Serviceability Limit State

New buildings, with Importance Level 2, are designed to remain serviceable after a small to moderate earthquake with a return period of 25 years. That is, any damage should be minor and easily repairable under such an extent. Services and equipment should be designed to withstand this level of shaking without damage. An assessment into the resilience and seismic restraint of services and equipment has not been investigated for the building at 314 The Terrace. An assessment of the building’s seismic rating is not required to consider serviceability requirements.

4.1.6 Building Access after an Earthquake

After a large earthquake, a well-performing building may be suitable for reoccupation soon afterwards, however local cordons and Civil Defence or City Council access restrictions may mean access to the building is restricted or prohibited. This has been a source of frustration for many Christchurch CBD tenants after the 2010 and 2011 earthquakes, whose buildings have otherwise performed well.

5 Conclusions and Recommendations

5.1 Building and Façade Condition

In general the interior and main structure of the building was found to be in a reasonable condition. The façade to the structure is in a particularly poor condition and needs to be either replaced or significantly refurbished.

In its current condition the façade is considered unstable and hazardous. In the event of high wind or a seismic event, it is likely that further deterioration and potential collapse of local areas of the façade will occur. It is also noted that continuing deterioration and corrosion will occur without prompt attention and repair.

Details of replacement and/or a new façade would likely need to be carefully considered to ensure the architectural intent and features of the façade are considered. Any new façade system not covering the exposed concrete slab edges and walls, may not completely eradicate potential on-going corrosion and associated spalling of the exposed concrete. Therefore, additional consideration and cost allowances may be required for long term maintenance of these concrete elements. Further, residual risks of potential falling debris/spalling concrete and unsafe balustrades and cladding panels will need to be managed with regular visual inspections and the like.

During the assessment of options (refer Appendix B), it should be noted that any significant modifications to the main transverse and longitudinal walls would require extensive re-engineering. Minor alterations such as limited penetrations to the walls could be accomplished however anything further would require extensive strengthening and modifications to the primary structural systems and is not recommended.

5.2 Piling and Sub-structure Condition

The construction method outlined in the Journal could lead to problems in achieving consistency of the aggregate and grout mix throughout the length of the pile and hence an uncertain foundation capacity. No evidence is available suggesting how the piling was monitored and how consistency in the pile concrete and integrity thereof was achieved. However, at the time of construction the Ministry of Works (MoW) would have likely known this and may have put in quality procedures to manage this, although this cannot be confirmed. We also note that there are no visible signs of settlement on site which indicates the piles are working adequately to support vertical loading.

Considering the above we believe it is necessary to test the piles for concrete quality and reinforcement to allow their construction to be understood. Further, testing would confirm the method of piling used for the building.

Testing would require breaking out at least two piles from the main structure and drilling through their centre to establish their integrity and construction. Excavating to the side and breaking out the concrete to the pile, pile cap and base of the shear wall would indicate the reinforcement used, as well as its connection to the superstructure. The testing would be difficult to achieve due to the position of the piles beneath the building and specialists in this type of testing would be needed to be engaged. The number of piles investigated may need to be increased depending on the findings as the investigations progress.

5.3 Detailed Seismic Assessment

Based on our assessment, we consider the building at 314 The Terrace to achieve less than 34%NBS and it is therefore Earthquake Prone. This corresponds to a grade D building as defined by the NZSEE guidelines and exposes the occupants to a high risk relative to a new building. The assessment is limited by the

condition of the façade which has deteriorated significantly enough that the capacity of parts of it cannot be relied upon.

If the façade is retrofitted the next most critical element achieves at least 50%NBS. This is governed by the tension yielding of the reinforcing bars in the longitudinal spine wall. We are unable to comment on the capacity of the pile foundations with the information available. If, upon investigation and analysis, the foundations score lower than the superstructure, the %NBS of the building will need to be reduced.

We understand it is VUW's policy that all its buildings achieve a minimum of 80%NBS (*as reasonably as is practicable*). Therefore, our recommendations are focused on achieving this outcome. Appendix B refers to the steps required to bring the existing structure to a minimum of 80%NBS.

Understanding the reinforcing content and homogeneity of the pile concrete is crucial in determining how the building will perform in a significant earthquake. Hence, we consider testing and investigation of the substructure is required of investigation required. It is possible the results of this investigation could impact the score the building achieves.

Investigation into the orientation and quality of the reinforcement to the spine wall to confirm the as-built structure matches the drawings and reaffirm the seismic performance of the superstructure.

5.4 Recommendations and Conceptual Strengthening

It is understood that Victoria University are also seeking advice on a number of uses for the building and the corresponding indicative scope of work required for the options. Appendix B provides an overview of the works required to bring the structure (as reasonably as practicable) to current code compliance.

We understand the options to be as follows:

- Refurbishment of the existing building back to its original condition (as reasonable as practicable) and compliant with current codes (as reasonable as practicable). It is understood that VUW's policy requires all property to target a minimum of 80%NBS and preferably 100%NBS.
- One additional student bedroom per existing apartment within the current footprint and general structural arrangement. It is understood that VUW's policy for student accommodation requires all property to target a minimum of 80%NBS and preferably 100%NBS. This would apply to student accommodation. Generally all previous seismic strengthening schemes for VUW student accommodation have achieved 100%NBS or greater, for example 132 The Terrace, 175 The Terrace and 100 Boulcott Street.
- Conversion to an office building (possibly open-plan).

The structure acts as a reinforced concrete shear wall structure and any modification to the walls, such as large openings, will require considerable structural alterations over and above the strengthening works noted to achieve 100%NBS.

From the results of the overall building condition survey and the Detailed Seismic Assessment, we recommend the following works to the building to prevent further deterioration and increase its seismic performance:

- Full replacement of the façade or significant repair and reinstatement of the existing.
- In addition to the façade replacement, we recommend that defective and/or carbonated concrete is removed and reinforcement that is found to be corroded or missing should be cleaned and/or replaced as appropriate. New concrete repairs are recommended to all affected areas. To assess the extent of works required for the concrete repairs, carbonation testing (likely to be by full scaffold access) of the façade will be required.

- Investigation into the piles by breaking out and drilling through the full depth of 2 existing piles (as noted above). Should defects or particularly poor conditions be encountered, it may be necessary to test a further two piles to establish whether the defects are widespread. Ideally access to the piles would be from outside the structure. However, it is noted that this may not be possible and some internal investigations of the piles may be required. Some breaking out of the shear walls may be required to gain access over the top of the test piles.

Following the investigations potential additional works may be identified. These could include the following:

- The piles may require improvement if deficiencies are found during the investigations. This could be in the form of additional piling to the underlying Greywacke rock, complete with associated pile caps and connections to the existing shear walls. The works will likely require piling outside and potentially inside of the building and hence specialist equipment will be required due to the limited headroom and proximity of existing structure
- Investigation into the orientation and quality of the reinforcement to the spine wall to confirm the as-built structure matches the design drawings.

Appendix A

Property Condition Photographs



Building Structure Photographs



STRUC 01



STRUC 02



STRUC 03



STRUC 04



STRUC 05



STRUC 06

Building Structure Photographs (cont)



STRUC 07



STRUC 08

Appendix B

Concept Design

- Refurbish building
- One extra bedroom added

Concept Design – Refurbish Building

| Item Heading | Item Function | Design Description | Estimating Details | Builder's Works | Comments |
|--------------|---------------|--------------------|--------------------|-----------------|----------|
|--------------|---------------|--------------------|--------------------|-----------------|----------|

Building Structure

| | | | | | |
|------------------------------------|------------------------|---|---|--|--|
| Investigation works | Shear Walls | To determine and confirm reinforcement content and details to shear walls. | Investigate longitudinal spine wall to understand reinforcing content and detailing | | |
| Investigation works | Piles and ground beams | To determine the piling technique used, reinforcement content, concrete strengths and pile integrity. | Investigate the piles and pile caps for both reinforcing content and strength of concrete | | |
| Main Structure Strengthening works | | Dependant on the investigation works to the existing reinforcement content and details, some additional strengthening maybe required to achieve 80 to 100%NBS | Strengthening works may be in the form of additional shear walls alongside the existing longitudinal spine wall. This may involve the introduction of additional 200mm thick reinforced concrete walling over the majority of the existing wall. New connections of the substructure and reinforced concrete pile caps with potential widening of the elements to cater for the increased width of wall is also likely to be required. | | Strengthening works is dependent on further investigation and testing of the existing structure. |
| Substructure Strengthening works | | It is difficult to determine potential strengthening works required for the piling and ground beam arrangements. Should the piles and connections to the shear walls be found to be insufficient through intrusive investigations, strengthening works may be required. | Strengthening works, if required, could be in the form of additional bored piles to the greywacke bedrock (between 6 and 14m below ground level) and additional pile cap provisions. Strengthening to the pile cap / ground beam and walls could be in the form of additional concrete works of steel angle bracketry forming connections between the two elements along the length of the walls (potentially both transverse and longitudinal walls). | | Strengthening works is dependent on further investigation and testing of the existing structure. |
| Façade works (rehabilitation) | | The facades are in very poor condition and considered unsafe and hazardous. | <ul style="list-style-type: none"> ■ Complete rehabilitation of concrete. ■ Removal of corrosion and rust (clean back to bright steel) ■ Breaking out damaged and spalled concrete | | |

| Item Heading | Item Function | Design Description | Estimating Details | Builder's Works | Comments |
|-------------------------------------|---------------|---|---|-----------------|--|
| | | | <ul style="list-style-type: none"> ■ Treatment of existing reinforcement ■ Additional reinforcement ■ Concrete repairs ■ Coating system applied to façade | | |
| Façade works (complete replacement) | | The facades are in very poor condition and considered unsafe and hazardous. | <ul style="list-style-type: none"> ■ Demolish existing façade ■ Reconstruct façade using lightweight materials and in keeping with heritage intent of existing structure. ■ Repair all corrosion and spalled concrete to main structure (concrete floor slabs and walls). ■ Coating system applied to façade. | | The alternative is to provide a curtain walling system that covers the existing structure in full to prevent continual deterioration of the slab edges. We note that this is dependent on the heritage requirements of the façade replacement. |

Concept Design – Refurbish Building with 1 Extra Bedroom

| Item Heading | Item Function | Design Description | Estimating Details | Builder's Works | Comments |
|------------------------------------|------------------------|---|--|-----------------|--|
| Building Structure | | | | | |
| Investigation works | Shear Walls | To determine and confirm reinforcement content and details to shear walls. | Investigate longitudinal spine wall to understand reinforcing content and detailing | | |
| Investigation works | Piles and ground beams | To determine the piling technique used, reinforcement content, concrete strengths and pile integrity. | Investigate the piles and pile caps for both reinforcing content and strength of concrete | | |
| Main Structure Strengthening works | | Dependant on the investigation works to the existing reinforcement content and details, some additional strengthening maybe required to achieve 80 to 100%NBS | Strengthening works may be in the form of additional shear walls alongside the existing longitudinal spine wall. This may involve the introduction of additional 200mm thick reinforced concrete walling over the majority of the existing wall. New connections of the substructure and reinforced concrete pile caps with potential widening of the elements to cater for the increased width of wall is also likely to be required. | | Strengthening works is dependent on further investigation and testing of the existing structure. |
| Substructure Strengthening works | | It is difficult to determine potential strengthening works required for the piling and ground beam arrangements. Should the piles and connections to the shear walls be found to be insufficient through intrusive investigations, strengthening works may be required. | Strengthening works, if required, could be in the form of additional bored piles to the greywacke bedrock (between 6 and 14m below ground level) and additional pile cap provisions. Strengthening to the pile cap / ground beam and walls could be in the form of additional concrete works of steel angle bracketry forming connections between the two elements along the length of the walls (potentially both transverse and longitudinal walls). | | Strengthening works is dependent on further investigation and testing of the existing structure. |

| Item Heading | Item Function | Design Description | Estimating Details | Builder's Works | Comments |
|-------------------------------------|---------------|---|--|-----------------|--|
| Façade works (rehabilitation) | | The facades are in very poor condition and considered unsafe and hazardous. | <ul style="list-style-type: none"> ■ Complete rehabilitation of concrete. ■ Removal of corrosion and rust (clean back to bright steel) ■ Breaking out damaged and spalled concrete ■ Treatment of existing reinforcement ■ Additional reinforcement ■ Concrete repairs ■ Coating system applied to façade | | Residual risk of ongoing deterioration of the concrete. May cause ongoing health and safety issues with unsafe cladding and balustrading as well as potential for falling debris. |
| Façade works (complete replacement) | | The facades are in very poor condition and considered unsafe and hazardous. | <ul style="list-style-type: none"> ■ Demolish existing façade ■ Reconstruct façade using lightweight materials and in keeping with heritage intent of existing structure. ■ Repair all corrosion and spalled concrete to main structure (concrete floor slabs and walls). ■ Coating system applied to façade. | | The alternative is to provide a curtain walling system that covers the existing structure in full to prevent continual deterioration of the slab edges. We note that this is dependent on the heritage requirements of the façade replacement. |