Clendon Burns & Park Ltd
CONSULTING CIVIL & STRUCTURAL ENGINEERS

Our Ref 210090
15th May 2012

Attention Mr B Crestani

Programme Manager
HNZC Asset Delivery
Housing New Zealand National Office
WELLINGTON

Dear Bede

Re: Gordon Wilson Flats - Structural Condition of the Façade

Further to the meeting in our office on Monday 7th May with you and Alex Neal from Beca we now have concerns about the current structural strength of the facades to the Gordon Wilson Building. At this meeting we were given a copy of the recently completed "Exterior Inspection Report" prepared by Goleman Wellington Ltd which highlighted many areas over the building façade where there has been severe spalling of the concrete.

Concrete spalling is caused by corrosion of the reinforcement which is a result of chlorides diffusing through the concrete over a long period of time. In combination with moisture they form acid which in turn reacts with the iron in the reinforcement to form ferrous oxide (rust). The volume of the ferrous oxide is many times that of the original reinforcement and as it forms, it expands, causing the cover concrete to crack and eventually fall off. The remaining exposed reinforcement then corrodes at an even faster rate. The potential for this process, which generally occurs over a long period of time, is present in all reinforced concrete elements.

The process can be mitigated by ensuring there is adequate cover to the reinforcement, the concrete used has sufficiently high cement content and is properly compacted at time of construction. Post construction, the progression of corrosion can be further delayed by the application of suitable high quality paint systems which then require regular maintenance throughout the life of the building.

In May 2010 we presented a building condition report to Housing New Zealand which highlighted amongst other issues the problem of concrete spalling in several areas of the building façade. We stated that the removal of loose concrete was urgent as it was an ongoing health and safety issue and there was evidence many pieces had already fallen from the building onto public areas. We understand that HNZC then organised for this to be done as a matter of urgency. Our report further suggested that further inspections be carried out in order to gauge the rate of degradation of the concrete.

The Goleman report is the follow-up façade inspection that was verbally recommended. This recent inspection revealed that there were many new areas where concrete had spalled. There has been significant degradation of the concrete to the east and west facades from when the first inspections were carried out less than two years ago. In our experience it is typical that once this type of degradation starts it will be on-going and become increasingly more widespread until significant repair work is carried out.

In addition to this on-going problem of dealing with the pieces of concrete falling from the building these recent inspections have exposed an even more serious problem.

The east and west facades of each unit supported by a small central precast concrete column spanning between floors. These are held in position at the top and bottom by a steel pin located into a small galvanised steel socket. The column in turns secures horizontal precast concrete panels which span between it and the inter-tenancy concrete shear walls. The tops of these precast panels and the edge of the column support the window frame. The steel pin fixing detail for the precast columns is the key to the stability and strength of each individual unit's façade. Although difficult to accurately assess it is has been inadequately designed by today's standards providing only about 30-35% of the desired seismic strength when in as-designed condition.

The Goleman Report has highlighted several areas on both the east and west facades where there is severe spalling of the concrete to the ends of these small precast columns. In the worst cases there has been loss of concrete support to the precast panels and around the steel pin fixing. In this instance there is now very little restraint provided to those elements. Unresolved deterioration will continue with even less restraint being provided than what is presently there.

There is a high risk the worst affected columns and precast panels could be dislodged or even fall from the building during severe wind or moderate seismic events. In the worst cases it is even feasible that a large person falling heavily at a critical location would provide a sufficient force to dislodge elements. Failure of these elements could then lead to window frames then being at risk of coming loose.

Given that these do not appear to be isolated instances over the building it is reasonable to assume that all similar elements on all elevations will ultimately be at risk. Any remedial work that is contemplated should be applied to all similar building elements.

In summary as a result of these and previous investigations the following major issues need to be addressed.

1. **Seismic Strength.**

   A detailed assessment was carried out by Clendon Burns & Park Ltd and a report dated 10th August 2010 was submitted to HNZC. The assessment rated the current seismic strength of the building at about 58% NBS. The report contained recommendations for strengthening work to the lower levels of longitudinal shear wall which would increase is seismic strength to about 73% NBS. To strengthen the building to a higher level would require significant additional work to the remaining shear walls and foundations which in our opinion would not be cost-effective.

2. **Façade Failure**

   As stated in the body of this report the fixings to the small central pre-cast columns supporting the building façade as originally designed were only at about 30% of what would be required by today's design standards. In some instances corrosion of these key elements has advanced to a stage where there is no visible evidence of any remaining fixing support! Once the precast column support becomes ineffective then potentially the whole individual unit façade can fail. Given that this deterioration will continue eventually affecting all similar columns there is no option but to replace the entire front and rear façade. In its current state it presents a real danger to both the occupants and wider members of the public in the immediate outside environment. For this reason alone the building could be deemed "uninhabitable".

3. **Concrete Spalling**

   There has been significant on-going spalling of concrete which presents a significant hazard to persons in the immediate vicinity outside the building. The size of some pieces falling from the building could, if hitting someone, result in severe or even fatal injury. The two detailed inspections carried out indicate that this process is escalating. Immediate steps should be taken to isolate an area on the ground for a distance of four to five metres out around the entire perimeter with a suitably robust construction fence at least 1.8meters high or similar protection. A protective roof should be placed wherever this protective fence line is breached to provide access to or egress from the building. This includes all main ground floor doorways and front and back doors to the ground floor bed-sits.

4. On-Going Protection to the Building

Even though decision to decant the building has been taken, in addition to securing the building against unauthorised entry some further short term remedial work should be undertaken until the long term future of the building is decided. Identify the locations from the Goleman Report and then provide new temporary fixings to the small pre-cast façade columns that are not adequately secured at their top or bottom fixing locations. This is to ensure the safety of those working around the building during the next few months.

We hope this clarifies the situation and are happy to discuss any issues arising from this with you.

Yours Faithfully
R N Patton

Director
Clendon Burns & Park Ltd

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SITE INSPECTION REPORT

JOB:	320 The Terrace - Gordon Wilson Flats Structural Report	**JOB NO:**	210090
TO:	HCNZ	**ATTENTION:**	Mr R Hopkins
INSPECTOR:	Ray Patton		
INSPECTION DATE:	29/04/2010		

1 Introduction

This report was prepared at the request of Mr Roger Hopkins of Housing New Zealand. He requested that we carry out a visual inspection of the Gordon Wilson Flats at 320 The Terrace and indentify any aspects of the building and its services that may require significant attention. In particular we were asked to:

1. Review building structure to see if there are any significant structural issues including whether access ways to the individual units are safe.
2. Review site associated retained or cut walls to determine whether there is the potential for significant failure or collapse.
3. Advise whether there are any areas identified where weather tightness may be an issue and the potential causes/solutions.
4. Inspect and review the following to identify obvious significant maintenance or upgrade obligations:
   - Sewer and storm water disposal
   - Any other building services items that may be evident such as gas or electrical systems

Excluded from this inspection report are the following:

1. The external cladding to the stair towers at each end of the building which are being assessed and replaced under a separate contract
2. Seismic assessment of the overall structure which is the subject of a separate report.
3. Fire and associated egress requirements which are again the subject of a separate report prepared by HNZC Fire Engineers
4. Overall compliance with the Building Code except where specific issues have been identified.

Ray Patton from Clendon Burns & Park Ltd carried out an inspection on Friday 23rd April 2010 in conjunction with Keith Johnstone Glen Wright and Simon Wills building services engineers from Stephenson & Turner Ltd. Stephenson and Turner's part of the brief was to report on the building services. These items are reported on under separate cover.

2 Building Structure

2.1 General

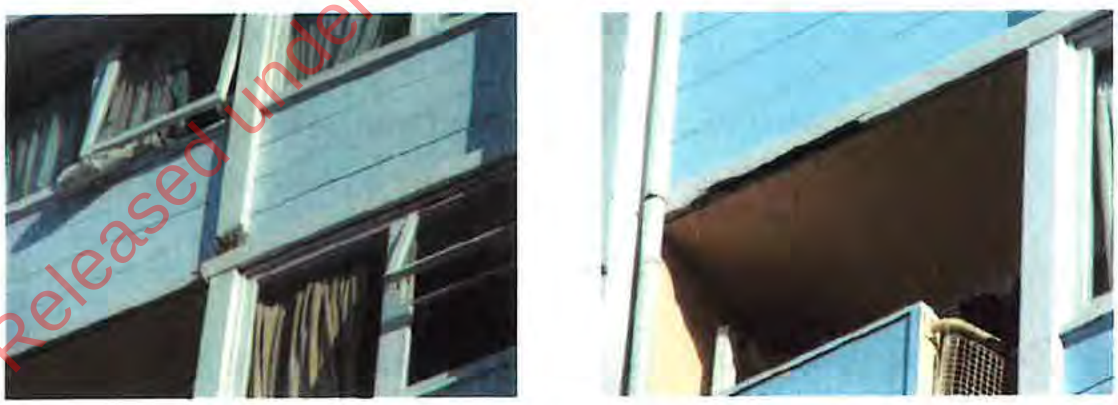
A review of the original structural drawings indicates that the building was designed in early 1954 and subsequently constructed making it approximately 55 years old. It was designed by the then Ministry of Works to their codes of practice that were current of the day. At this time the

MOW had developed their own codes that were generally more stringent than the corresponding Codes developed by Standards New Zealand and used by the private sector.

2.2 Spalling Concrete

The building is generally suffering from significant deferred maintenance. The most notable example of this are areas where concrete has or is about to spall as a result of corrosion of the reinforcement .

Front Façade - Concrete About to Spall



Front Façade - Concrete has Spalled

This process which in time affects most concrete buildings is a generally a result of chlorides penetrating the concrete and forming acid when combined with water. Eventually the acid will penetrate deep enough to react with the steel reinforcement. The resultant ferrous oxide (rust) is approximately five times the volume of the original steel and as it expands it causes the cover concrete to spall.

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Calcium chloride was sometimes added to a concrete mix to speed up its setting time and give early strength gain. It is now known that its addition can also accelerate the corrosion process. In particular calcium chloride was often used in precast concrete work where early strength gain was desirable and it is possible it could have been used on parts of this building.

This corrosion process can be delayed by:

- ensuring adequate cover to the steel at time of construction
- using higher cement content in the concrete mix at time of construction to provide an alkaline environment
- applying and maintaining a suitable paint system




Rear Façade - Examples of Spalling Concrete

There is the potential for pieces of concrete to fall from the building at any time. This poses a threat to both tenants and visitors to the building. Pieces are large enough and when falling from a significant height could cause severe injury or possible death to someone who is hit on the head. We note that there is an access path running along the front of the building and although the chances of being hit are low persons using this path are potentially at risk from falling debris. We note however that the areas of the building we concrete pieces have previously been dislodged are not generally located over the building entrances

Immediate steps should be taken to ensure that people are kept from congregating in the front or rear of the building for a minimum distance of five metres until the facades have been inspected and any loose concrete removed. This can be achieved by either scaffolding the building (which could be done in conjunction with re-painting which is referred to further on) or by suspending a swinging stage and working progressively over each façade.

More involved permanent repair work would then be required to ensure the ongoing integrity of the exposed concrete on each façade.

2.3 Painting the Building

As mentioned above one way of slowing the onset of reinforcement corrosion is to ensure the paint system is well maintained. Inspection of each building face shows the paint system has failed. It is interesting to note that in many places the paint layer has delaminated from the building but is still largely intact.

Failure of Paint System (Delaminating).

Although unable to inspect the paint work close up, from a distance it appears to be retaining water in some of the large bubbles. The most likely cause of this failure is inadequate preparation of the sub-base resulting in lack of cohesion between the concrete and the paint film. Before new paint is applied the problem of the deteriorating concrete needs to be addressed.

3 Other Structural Issues

The major problems have been discussed above but there are other items that need to be addressed

3.1 Rotten Balustrades to Rear Walk ways.

Rotting Top Timber Railing to Balustrade

The top timber railing to the balustrades on the rear walkways have rotted away in many places. This railing trims the top of the balustrade and also supports a pipe handrail which is fixed to the top of it at regular centres. This pipe hand rail also provides the means by which the hose reels on each level are supplied with water. Should the handrail fixings become free as a result of timber railing rotting away it is possible that the pipe s could be fractured resulting in loss of water supply.

The rotting top rail also calls into question of whether or not the timber framing to the balustrades has started to rot where it is fixed at its base to the concrete slab. As replacement top timber rail is well overdue due it would be prudent to remove the balustrade covering panels to check the frame base fixings at the same time.

3.2 Top Hung Windows

During our visual inspection of the front façade we observed that many of the opening windows are top hung sashes fixed with two hinges. Many of these hinges showed signs of rusting albeit relatively minor in many cases.

Minor rusting to Window Hinges.

All windows should be inspected from inside the flats to confirm the level of corrosion to the hinges and their fixing screws. Any showing evidence of corrosion should be replaced. If left to badly corrode the action of opening the window could cause it to come of its hinges dropping to the ground below.

3.3 Stair Access to the Lift Machine Room

Access Stair to Lift Machine Room Badly Corroded

The access stair is badly corroded with parts of the stringer completely rusted through. This should be replaced at the earliest opportunity and workman using advised of the risk especially if more than one person is on the stair or a heavy load is being carried up it.

3.4 Roof Fabric

The roof is at two different levels The lower roof covers the top of building immediately above the flats and provides an area for drying laundry. Along one side running the full length of the building is a penthouse which consists of many rooms proving both storage and laundry washing and drying facilities. This roof of the penthouse extends over the stair wells at each end.

The lower roof appears to be in good repair having a reinforced membrane to sustain the foot traffic and appears to have been well maintained over the life of the building. The upper roof however appears to be the original and has reached the end of its useful life.

When the stair towers at each end of the building are being refurbished it is recommended that this roof be replaced.

Lower Roof is Satisfactory

Upper Roof Needs Replacement

Note The beer bottles in the gutter in the right hand picture which implies access to this dangerous area of the roof is not secure.

3.5 Protective Fence to Laundry Area

Palings missing from Security Fence

This fence provides the only protection from falling off the laundry drying roof area. While generally in good condition there are a some palings missing. It is unclear what has caused their removal. However it is possible they have rotted out. The whole fence should be carefully inspected and all unsatisfactory palings replaced.

3.6 Sprayed Concrete Retaining Wall at Rear of Site

Prior to the building being constructed a large sprayed bank was created behind to facilitate the creation of a relatively level construction platform.

Many weep holes placed over the plane of the wall to drain any water that may seep out of the bank. It is important that these a re maintained by periodic cleaning out including the removal of any live plants. Also it was observed that there are some cracks in the concrete out of which various plants including small pohutukawa trees. These should be removed and areas treated to inhibit future re-growth.

Sprayed Concrete Bank
Greenery to be removed and Weep holes Cleaned

3.7 Young Trees Growing on Building

During our inspections we noted that small young trees were growing in various crevices on the building. These should be removed as soon as possible as their expanding root systems will eventually cause damage to the concrete causing splitting and spalling.

Examples of Greenery growing in Building Crevices

Conclusions

The building generally suffering from much deferred maintenance. We have highlighted some items that require urgent attention and others that require attending to in the short term (6 -12 months time frame) or medium term (12-18 months).
These are :

Urgent Attention
- Item 2.2 - loose concrete
- Item 3.1 - rotten balustrade rails
- Item 3.3 - access stair to lift machine room
- Item 3.5 - protective fence to laundry area

Short Term Attention
- Item 2.2 - long term remedial work to concrete
- Item 2.3 - re painting building
- Item 3.2 - inspection of all window hinges in particular top hung windows

Medium term

- Item 3.4 - Replacement of penthouse roof fabric
- Items 3.6 - Clean up of sprayed concrete wall
- Item 3.7 - Removal of greenery from building crevices.

We are happy to further discuss any aspects of this report

Report prepared by
R N Patton ME MIPENZ CPEng IntPE
Director.

Clendon Burns & Park Ltd
CONSULTING CIVIL & STRUCTURAL ENGINEERS

Structural Report

Detailed Seismic Assessment
Gordon Wilson Building
320 The Terrace
Wellington

Date 10 August 2010

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NOTE:

It is understood that this Report has been prepared at the request of Housing New Zealand Corporation to be used for their purposes only, and neither Clendon Burns & Park Ltd nor any of its Employees accept any responsibility on any ground whatsoever to any other party or person who relies upon it.

1. INTRODUCTION

This report has been prepared at the request of Housing New Zealand Corporation.

In May 2006, the Wellington City Council (WCC) adopted a policy for dealing with Earthquake Prone Buildings within its jurisdiction. Under this document all buildings that were built or strengthened to pre-1976 structural design codes, are to be assessed for their strength to resist earthquakes. This is done by using the Initial Evaluation Procedure (IEP) as set out in the New Zealand Society for Earthquake Engineering (NZSEE) publication 'Recommendations for the Assessment and Improvement of the Structural Performance of Buildings in an Earthquake'. Modifications to the Council policy were made in 2009.

An IEP prepared by Clendon Burns & Park Ltd identified this building to be Potentially Earthquake Prone.

An adjunct to the IEP and a more accurate method of evaluating the current seismic strength of a building is to carry out a in-depth detailed assessment. This involves modelling the building with a 3-D computer programme to ascertain seismic loads and resultant stresses when these loads are applied. The strength of Individual structural elements are then compared with what would be required by current codes of practice leading to an evaluation of the overall strength of the building.

2. EXECUTIVE SUMMARY

The detailed assessment of the Gordon Wilson Flats structure indicated that, in terms of the definition of "earthquake-prone" within Section 122 of the Building Act the building is not earthquake prone but can however be categorised as earthquake-risk. In the event of a moderate to severe earthquake this means that although the building is unlikely to collapse it would suffer significant damage rendering it uninhabitable.

- The analysis found that globally the building achieved 58% NBS. This was defined by the lower levels of the longitudinal shear wall being the weakest structural element. The building is therefore classified as a grade "C". The definition of the grading system is shown in Table 1.

- It is recommended that raising the level to 73% NBS could be achieved by strengthening the longitudinal shear wall between ground and 3rd floor levels. This would rate the building to a level greater than 67% NBS which is the recommended minimum proposed by the New Zealand Society for Earthquake Engineering and the Wellington City Council. There is no point in strengthening beyond this level as other structural elements would also then require strengthening.

3. EXISTING STRUCTURE

3.01 General

The building was designed by the Ministry of Works, Architectural & Structural Divisions and constructed in 1959 making it approximately 50 years old. A full set of structural drawings were available for reference during this analysis. It is assumed as a minimum standard that the building was constructed in accordance with these documents.

3.02 Existing Structure

The building is a twelve storey block of flats with a roof laundry building along the west side of the roof and ground floor studio flats at ground floor level. In between there are located five levels of two storey flats, each unit being bounded by concrete shear walls on either side and a concrete floor slab above and below. This results in a structure having a single longitudinal spine shear wall with sixteen transverse shear walls and concrete diaphragm floors occurring only at each alternate level. The flats are two storey with a timber floor in between. The ground floor studio flats have a concrete floor above and suspended timber floor below founded on a concrete structural ground slab.

Each end of the building has a stairwell, glazed on two sides with a concrete wall on the east side. A separate lift tower is situated on the west side of the building. The lift tower is linked to the main building with concrete floor slabs back to the main building at every second level.

The building is founded on driven 16” (400mm) octagonal piles. These are grouped with 16 piles per transverse shear wall. The outer six piles to each end of each transverse wall are splayed out. Pile lengths vary from 20’ to 45’ long (6.0m – 14m). The lift tower is on an isolated piled slab. The outer foundation to the end stairwells is a strip foundation. The longitudinal shear wall sits on a ground beam running the length of the building. It is only piled at each transverse shear wall location.

Appendix A contains a selection of original plans (Figures 2-5).

3.03 Ground Conditions

The original drawings include drawing 17a (Figure 2 – Appendix A) which shows soil bore logs from 8no. boreholes. These indicate a maximum borehole depth to 53’ (16m) which pass down through brown gritty clay, soft blue sandy silt, dark brown silt with signs of vegetation, very gritty clay with angular gravel and clean greywacke. From these bore logs the site soil category of Class “C” (shallow soil) was inferred which was then used in determining seismic loads.

3.04 Previous Modification to existing building

A visual external site inspection indicated that there have been no structural modifications that would modify the seismic behaviour or the strength of the building.

3.05 Existing Structural Condition

A non-invasive detailed survey of the structural condition of the existing building was carried out. There are minor durability issues for the structure. During the walkover inspection it was noted there was evidence of spalling of exterior concrete around some window openings and at the exposed end of the transverse shear walls. These would have no significant effect on the overall basic structural integrity of the building.

4. SEISMIC STRENGTH REQUIREMENTS

4.01 Current Seismic Design Approach

Since the building was designed, in 1959, seismic codes and philosophies have been changing and evolving. Modern design philosophy requires a building to be designed with the ability to deform to absorb earthquake energy, without significant loss of strength due to sudden failure of critical structural components, most notably vertical load carrying elements. This desirable flexibility is called "ductile" behaviour and is achieved by appropriate detailing as required by current design codes. This knowledge has been incorporated in the 1976 Loadings Code and its subsequent updates in 1984, 1992 and 2005. At similar times the associated material codes have also been appropriately updated.

4.02 Building Act 2004

On 21 February 2005, the regulations were tabled, defining how earthquake risk buildings are to be defined. These are:

Section 122 of the Building Act 2004, defines the meaning of Earthquake-prone Buildings:

(1) A building is **earthquake-prone** for the purposes of this Act if, having regard to its condition and to the ground on which it is built, and because of its construction, the building
  (a) will have its ultimate capacity exceeded in a moderate earthquake (as defined in the regulations); and
  (b) would be likely to collapse, causing:
    (i) injury or death to persons in the building or to persons or any other property; or
    (ii) damage to any other property.

(2) Sub-section (1) does not apply to a building that is used wholly or mainly for residential purposes unless the building:
  (a) comprises 2 or more storeys;
  (b) and contains 3 or more household units.

Earthquake Prone Buildings - Moderate Earthquake

For the purposes of Section 122 (meaning of earthquake-prone building) of the Act, moderate earthquake means in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking that would be used to design a new building at that site.

Earthquake Risk Buildings - Moderate Earthquake

Earthquake Risk Building is regarded as applying to any building that is not capable of meeting the performance objectives and requirements outlined in the NZSEE document 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes'. This identifies a category of building which lies between Earthquake Prone Building (at 33% NBS) and 66% NBS. This acknowledges that there is still a significant risk involved to buildings within this performance range.

There is no legislated requirement to upgrade Earthquake Risk Buildings, however due to the significant risk involved NZSEE and WCC strongly recommends that every effort be made to achieve improvement to at least 67% NBS.

NZSEE grading scheme

In addition to the legislative requirements set out above, the NZSEE is developing a scheme where buildings are given grades to reflect their ability to resist earthquake loads.

If introduced into the property market, it will raise awareness of the risk from earthquake on buildings. Owners of higher rated buildings will have lower insurance premiums and a more marketable property. This should have the effect of forcing owners of buildings with low grading to upgrade them. This beneficial result of this will be in increase of the earthquake strength of the building stock generally thereby reducing casualties in the event of a major earthquake .

The following table, extracted from NZSEE document 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes', sets out the proposed grading system.

Percentage of New Building Standard (%NBS)	Letter grade	Relative risk (approx)	NZS 4203: 1976 or better	1965 - 76 No CSWs	1935 - 65 No CSWs	2/3 Chapter 8	Buildings with CSWs
>100	A+	> 1 time					
80 - 100	A	1 -2 times					
50 - 80	B	2 - 5 times					
33 - 50	C	5 - 10 times					
20 - 33	D	10 - 25 times					
<20	E	> 25 times					

Table 1: Grading System for Earthquake Risk

Notes:

1) % NBS is the percentage New Building Standard score for a particular building

2) Values shown for % NBS for building groups are indicative only and will vary with location, assessed ductility, features. Many buildings may have been designed for more than the minimum requirements of the standards of the day.

3) Letter grade is an indicator of likely performance in earthquake.

4) Relative risk (RR) is the ratio of probabilities that the ultimate strength will be exceeded in any given period of time, i.e. RR = (probability for existing building with % NBS value shown) ÷ (probability for building with 100% NBS).

5) CSW stands for critical structural weaknesses.

Critical structural weaknesses are elements or characteristics of the building that make it more vulnerable to severe damage or even collapse during an earthquake. Critical structural weaknesses generally occur in buildings designed before the 1976 Loadings Code come into effect, where a previously accepted practice is now considered to be unacceptable as engineering knowledge of earthquakes has developed.

It should be noted that the level of risk is not linearly proportional to the strength of the building. The risk that the ultimate strength of the building will be exceeded in any given period decreases exponentially with increasing strength. This can be seen in the Relative Risk (RR) column of Table 1.

4.03 Current Design Codes

The current Design Codes relevant to this assessment are:

- NZS 1170.5:2004 Structural Design Actions
- NZS 3101:Part 1:2006 Concrete Structures Standard

4.04 Wellington City Council Requirements

The Wellington City Council (WCC) currently has a policy in place to implement these sections of the Act, covered in the WCC document “Earthquake Prone Buildings Policy – 2009”.

The WCC Policy requires that a building must comply with the requirements of Section 122 of the Building Act. A building is earthquake prone if it has strength less than 34% of the seismic loading standard NZS1170.5:2004. A building which is assessed to be earthquake prone will need to be strengthened within a time frame in line with the WCC Policy.

4.05 Heritage Buildings

The Gordon Wilson Building has been identified as a Heritage Building on the Wellington City District Plan (Figure 1 - Appendix A). The Building Act requires that Councils must ensure that all earthquake-prone buildings are strengthened to at least meet the minimum prescribed standard as detailed above, or be demolished. However the policy approach of Wellington City Council towards Earthquake Prone Building which are also Heritage Buildings is to reduce the impact of any strengthening work required on the heritage fabric of the building by:

- Strengthening to a minimum level so that it is no-longer earthquake-prone.
- The maximum time frames for strengthening work will apply, just as it does to all buildings.
- A management plan outlining how strengthening will preserve the heritage fabric of buildings is to be provided.
- Demolition is not encouraged.

Although the Gordon Wilson building is not Earthquake-Prone if any strengthening is undertaken it will need to comply with the above council requirements.

5. CURRENT EARTHQUAKE STRENGTH OF THE BUILDING

5.01 Detailed Assessment

A detailed desk top assessment of the existing building was carried out based on the existing structural and architectural drawings. The analysis was based on the principles outlined in the NZSEE ‘Assessment and Improvement of Structural Performance of Buildings in Earthquakes’. No invasive site investigation was carried out to determine of the actual strength of the materials.

This desk study is intended to provide a building strength in terms of “percent new building standard”, % NBS, relative to NZS1170.

5.02 Methodology of the Detailed Assessment

In assessing the capacity of the structure the following procedure and assumptions were used:

- The building seismic design loads were assessed using relevant clauses of NZS 1170.5: 2004. This gave class C for the subsoil ground conditions.

- A an Importance Level of 2 and a Return Period Factor of 1.0 was used. This is considered appropriate for a 12 storey block of flats.

- The building was modelled in ETABS, three-dimensional analysis software, with stiffness reduction factors applied to elements as recommended by the concrete design code NZS 3101, to obtain the earthquake response of the building. Figure 1 Appendix B shows a 3D view of the ETABS model used in the detailed assessment.

- An overall ductility factor of μ=1.00 was used for stability with a structural performance factor of 1.0.
- A ductility factor of μ=1.25 was used for the strength design of both shear walls.
- A ductility factor of μ=1.25 was used for the strength design of all remaining elements.
- Seismic weights were assessed from the existing structural plans.
- An equivalent static analysis was performed to obtain the seismic forces on the building.
- Probable strengths for the major structural elements were assessed from the available information and drawings.
- These strengths were compared to the design actions obtained from ETABS to determine the capacity of the structure, and therefore the overall rating in % NBS.

5.03 Findings

The findings from this detailed assessment are as follows:

- The fundamental period of building is 0.52 seconds in the transverse and 0.4 seconds in the longitudinal directions. As this period is 0.4s the building attracts close to the maximum seismic load in the longitudinal direction and about 85% in the transverse direction.
- The inter-storey drifts are less than the allowable values in the current loadings code NZS1170.5. In addition, this building is well separated from the property boundaries and any adjacent buildings.
- The assessment indicates that, in terms of Section 122 of the Building Act's definition of "earthquake prone", the building is not an earthquake prone building.
- The analysis found that the global stability of the building achieves greater than 100% NBS.
- The analysis found that the shear capacity of the driven piles achieves 77% NBS.
- The analysis found that the longitudinal shear wall achieves not less than 100% NBS above 5th floor level. From 3rd floor to 5th floor levels the wall achieves 79% NBS. From 1st to 2nd floors, the wall achieves 63% NBS. From ground to 1st floor levels the wall achieves 58% NBS.
- The analysis found that the end transverse shear walls achieve not less than 100% NBS above 3rd floor level. From 1st to 2nd floor the end transverse walls achieve 89% NBS and from ground to 1st floor 94% NBS. The internal transverse walls achieve 73% NBS at 3rd floor level. From 1st to 2nd floor not less than 100% NBS and from ground to 1st floor 99%NBS
- The lift tower is connected to the main building with concrete slabs at each level. The 1st floor level slab achieves 92% NBS. The other slabs above all achieve not less than 100% NBS.
- It is proposed that the longitudinal spine wall from ground floor to third floor levels (58%) be strengthened to bring the whole building up to a 75% NBS without the need for further strengthening to other areas of the structure. The next critical elements are the internal transverse shear walls at 3rd level (73% NBS) and shear in the piles (77% NBS)
- The results of the analysis have been appended for reference (See Appendix B).

5.04 Earthquake Strength in terms of Building Act 2004

As discussed earlier, Section 122 of the Building Act 2004 defines whether a building is defined as earthquake prone.

The Detailed Analysis has determined that the building is not Earthquake Prone, but is however Earthquake Risk.

In terms of the NZSEE Grading System the building grade is assessed to be Grade “C”. The building is not considered a ‘high risk’. In line with the NZSEE recommendations, it is recommended that the building be strengthened to a minimum of 67% NBS.

6. STRENGTHENING OPTIONS

6.01 Strategy for improving structural performance

Although the legal minimum performance of a building is 34% NBS, NZSEE strongly recommends a building be brought “as near as reasonably practicable” to that of a new building. Ideally any building should be brought up to 100% NBS. However it is recognised that this is not always practical. and 67% NBS is seen as an acceptable level of risk. This level also offers some future proofing against any further changes to the Building Act.

In this particular case, the building may be brought up to 75% NBS by strengthening the spine wall between ground floor level and 3rd floor level. This may be achieved by providing a 150mm thick sprayed concrete shear wall fixed to one side of the existing spine wall.

Provisional drawings detailing the scope of work are included in appendix B

7. CONCLUSIONS

A desk study of the Gordon Wilson Building at 320 The Terrace, Wellington was carried out and the findings are as follows:

In terms of Section 122 of the Building Act’s definition of “earthquake prone”, the building is not an earthquake prone building. The building is Earthquake Risk.

In terms of the NZSEE Grading System, the building strength is assessed to be 58% of the new building standard in certain main structural elements and hence it is classified as grade “C”. The definition of the grading system is shown in Table 1.

Strengthening up to 75% NBS could be achieved by strengthening the longitudinal shear wall from ground floor up to third floor levels.

8. RECOMMENDATIONS

As the building’s strength is not below the legal minimum, strengthening will not be required. However it is recommended that consideration be given to improve the seismic strength of the building to at least two-thirds of the current code. This would involve strengthening to the longitudinal shear wall between ground floor and level 3.

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Report prepared by:

Peter Johnson
Design Engineer

CLENDON BURNS & PARK LTD

Report approved by

Ray Patton
Director
ME (Civil), MIPENZ, CPEng, IntPE
CPEng No. 026288

9. APPENDIX A - PLANS

- Figure 1: Wellington City Council District Plan Map
- Figure 2: Original borehole logs. (Drawing 17A)
- Figure 3: Foundation Plan (Drawing 7)
- Figure 4: Building Plans
- Figure 5: Building Sections

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Figure 1: Wellington City Council District Plan Map

Figure 2: Original borehole logs. (drawing 17A)

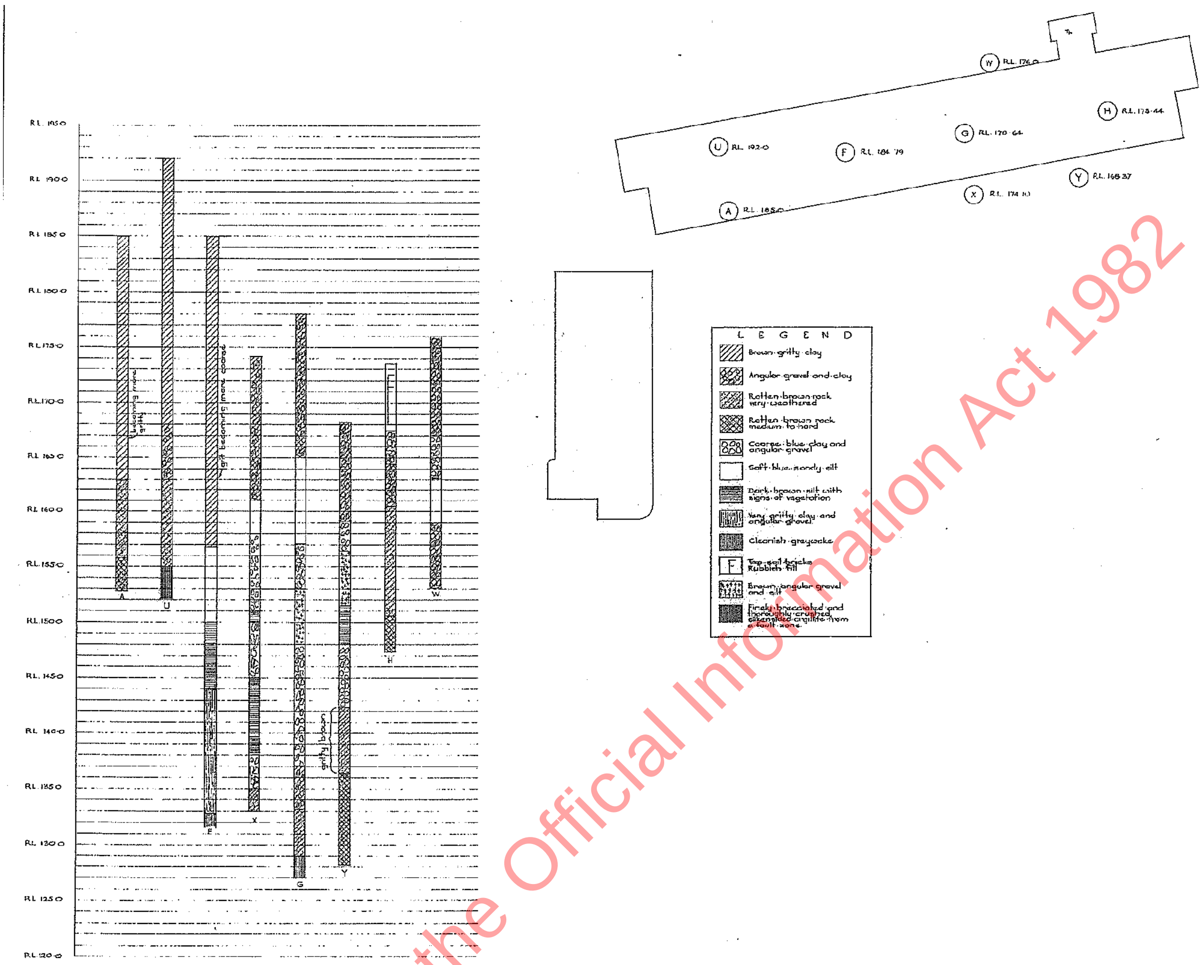

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Figure 3: Foundation Plan (Drawing 7)

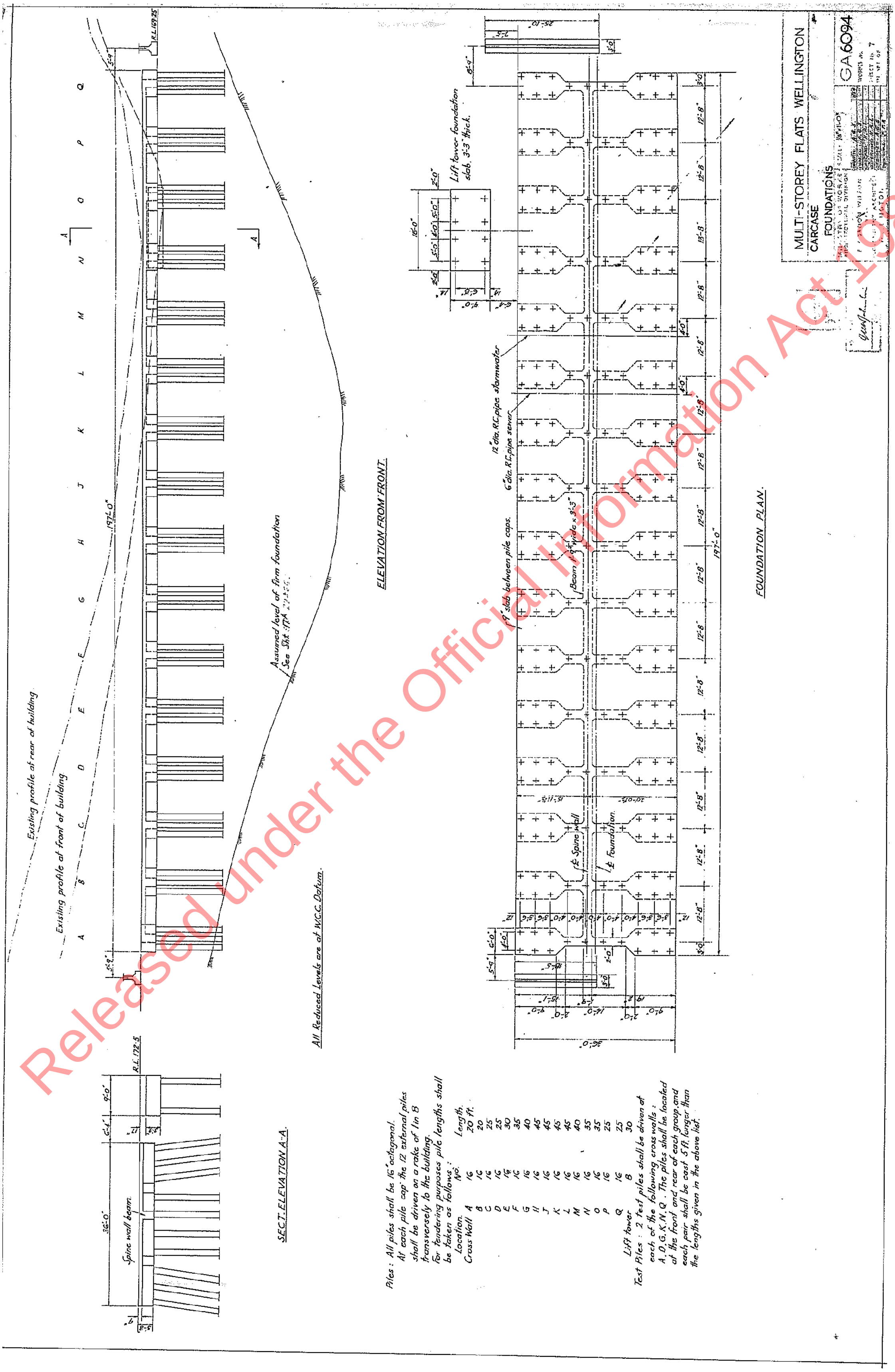

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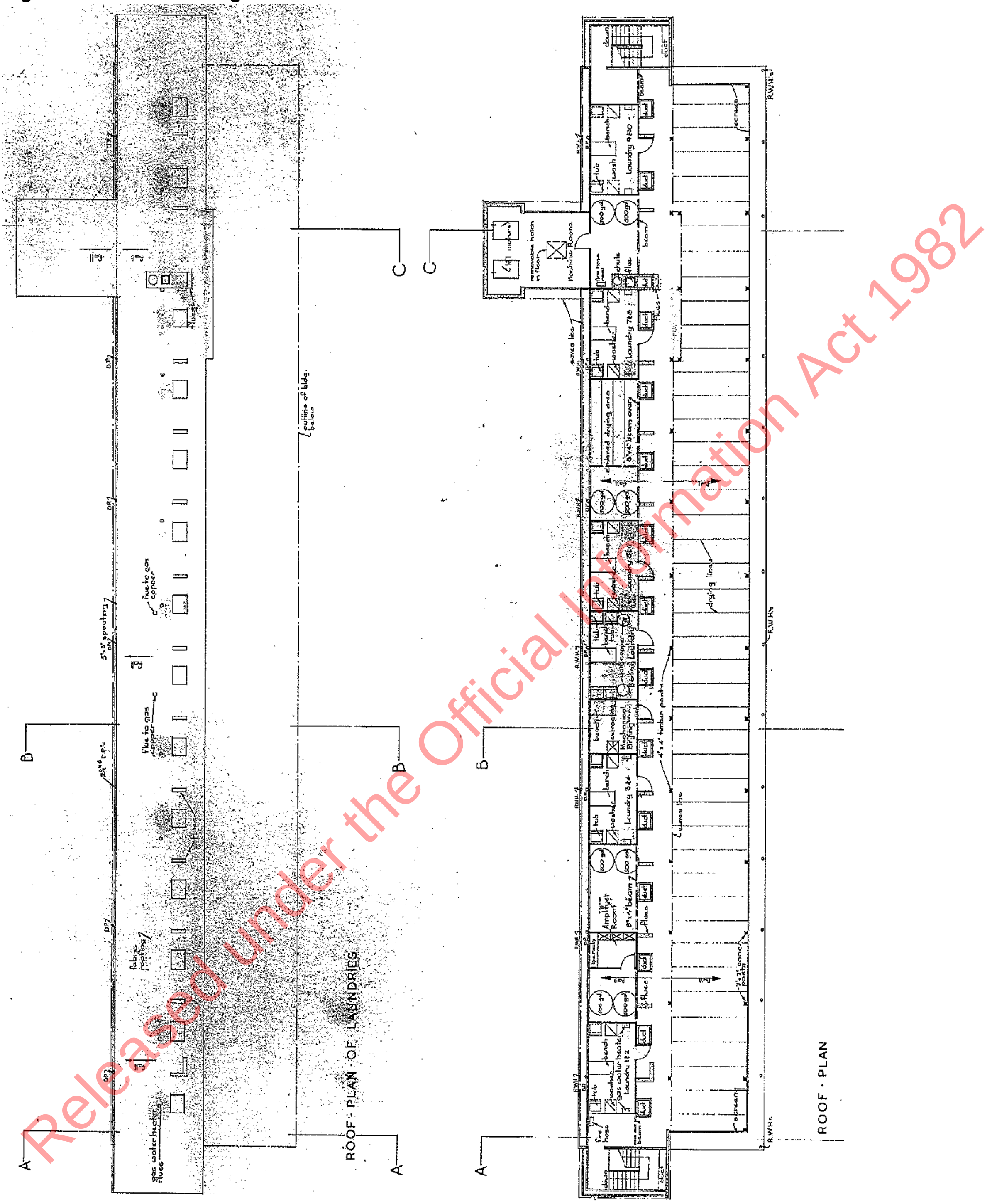
Figure 4: Building Plans



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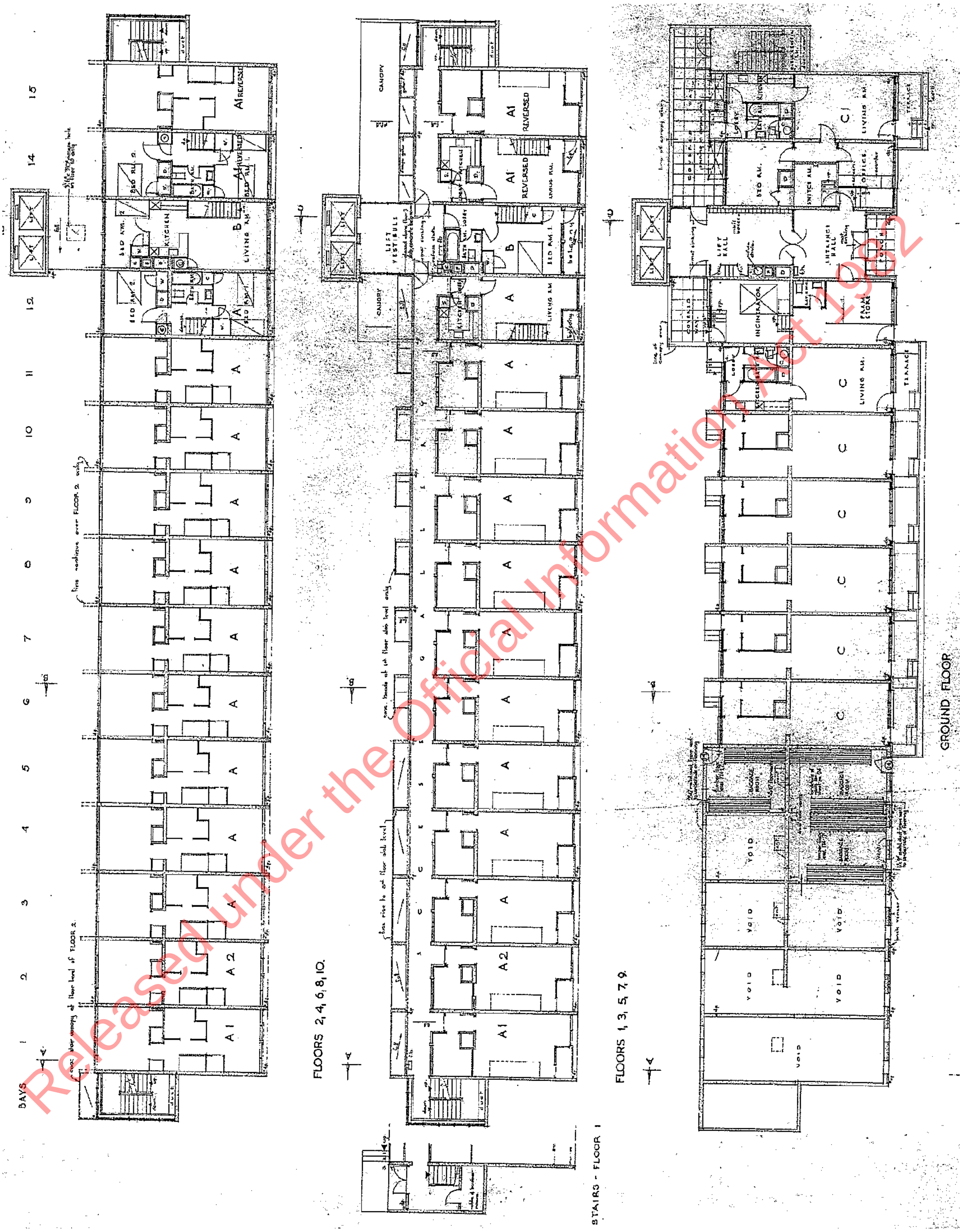

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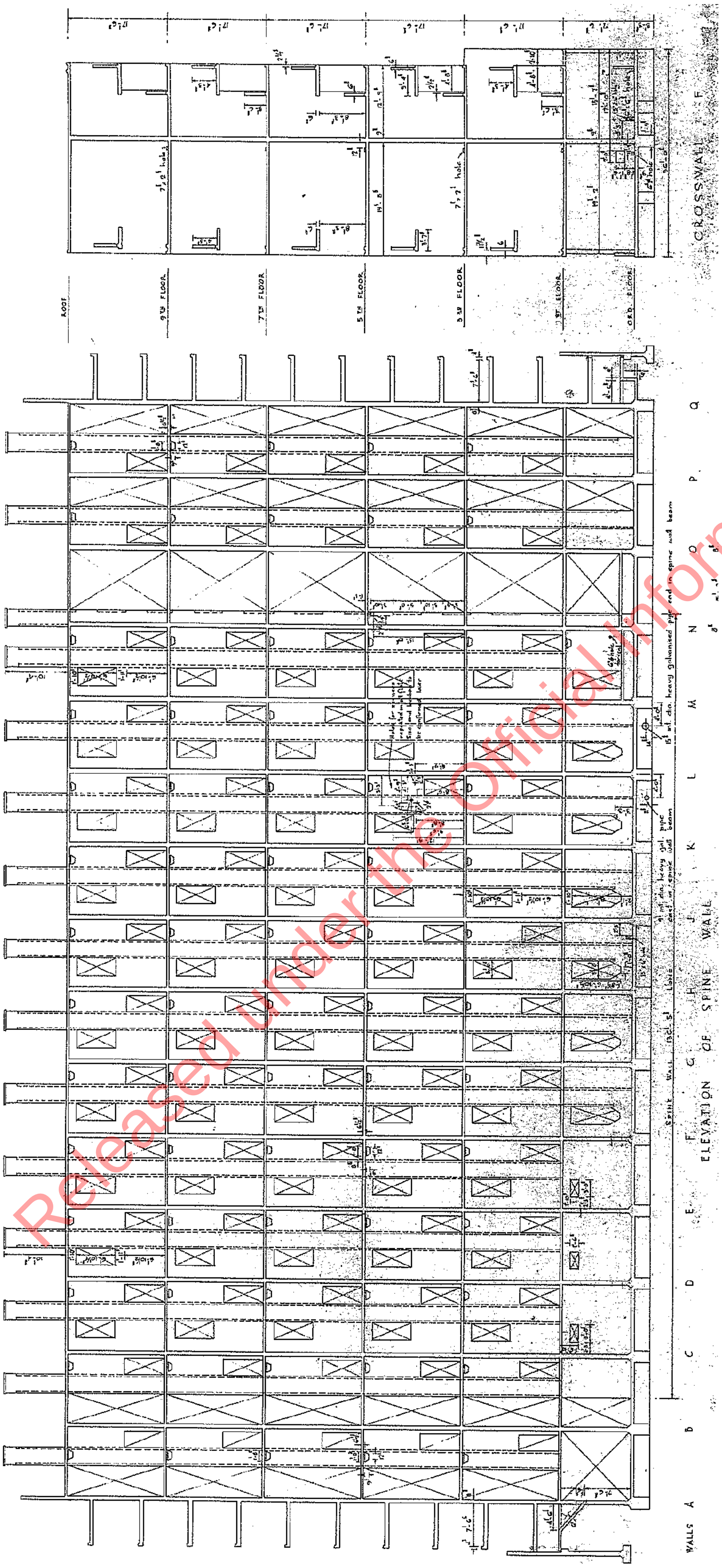
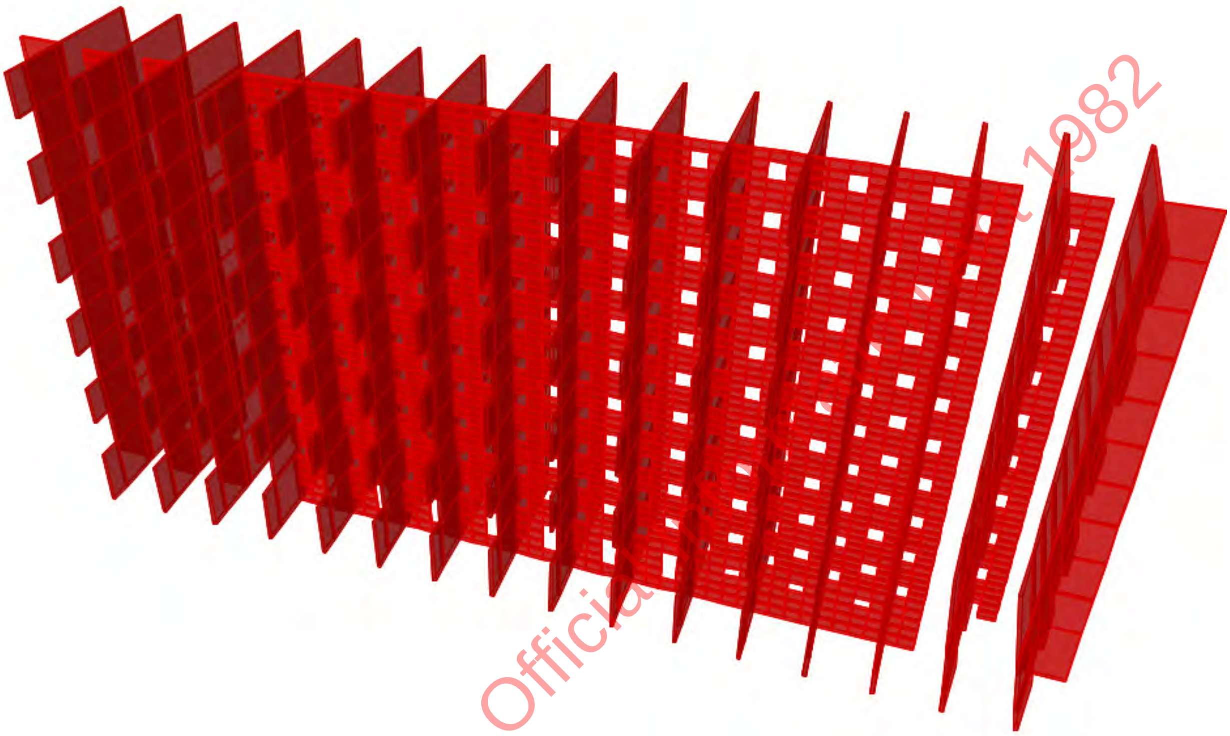
Figure 5: Building Sections



10. APPENDIX B - CALCULATIONS



Etabs Model Showing Tranverse and Logitudinal Shear walls

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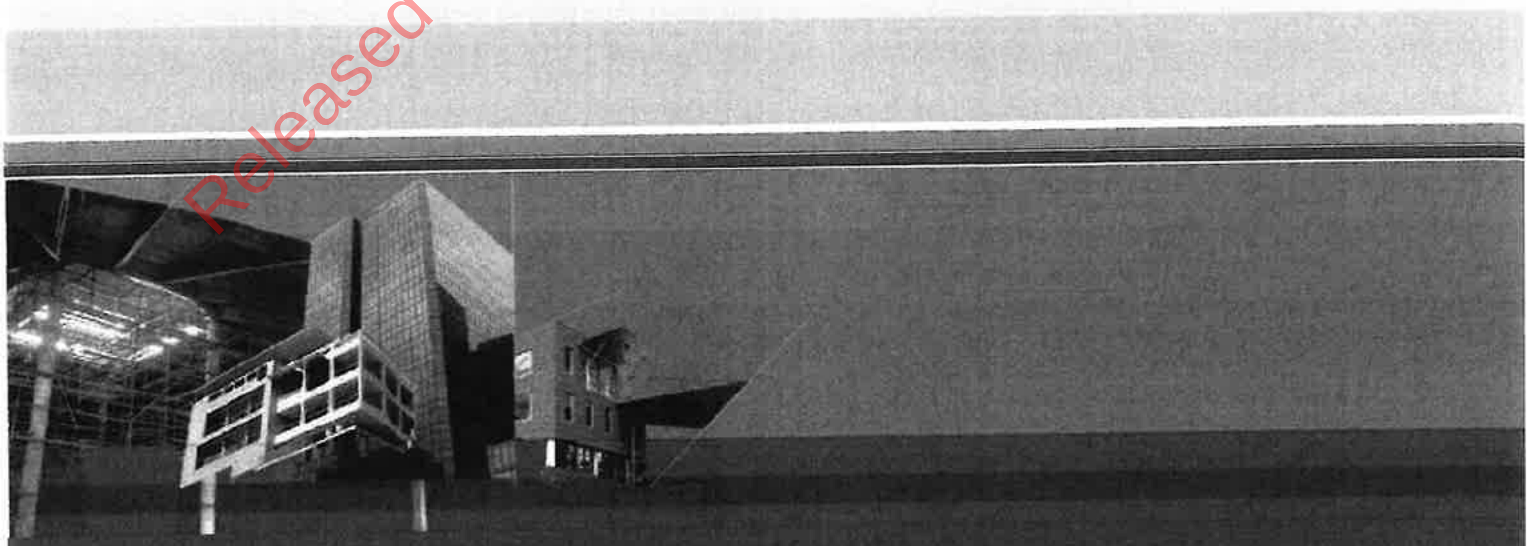
Structural Report

IEP Report
Gordon Wilson Building
320 The Terrace
WELLINGTON

Date **May 2010**



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1. Index

2. Introduction

Building Name: Gordon Wilson Flats

Address: 320 The Terrace, Wellington

Owner: Housing New Zealand Corporation

This building has been assessed by the Wellington City Council as Potentially Earthquake Risk but not Potentially Earthquake Prone. The purpose of this IEP is to review the WCC evaluation but in more depth by referring to existing drawings and inspecting the building inside and out.

3. Executive Summary

Our evaluation has revealed that the Wellington City Council has made some erroneous assumptions with its IEP leading to their conclusion that the building has a greater seismic strength than it actually has.

We believe the following assumptions made by the WCC are in error

- They have assumed that the framing system in the longitudinal direction consists of moment resisting concrete frames (MRCF) with an inferred period of 1.24 seconds. Whereas the lateral system is one long squat shear wall of low ductility having a period of about 0.4 seconds. This significantly increases the seismic forces in the longitudinal direction
- They have assumed that the building is founded on rock (Soil category A or B) . However it is founded on a piled system with piles up to 13 metres deep. This means it is more likely a type C soil. This also increases the design seismic loads on the building with a consequent decrease in its strength.
- WCC do not appear to have picked up that there are reinforced concrete floor only at every second floor. The intermediate floors are timber construction. This will likely impact on the stability of the shear walls and hence their seismic strength as they are only effectively restrained at every second floor.

These variations have a significant effect on the buildings assessed seismic capacity.

	%age New Building standard Longitudinal	**%age New Building standard Transverse**
WCC Evaluation	**60.4**	**40.9**
CBP Evaluation	**16**	**37**

Our conclusion is that the building is Potentially Eartquake Prone rather than Potentially Earthquake Risk by virtue of weakness in the longitudinal direction.

A detailed full assessment should now be carried out to determine the actual current strength of the building.

4. General information

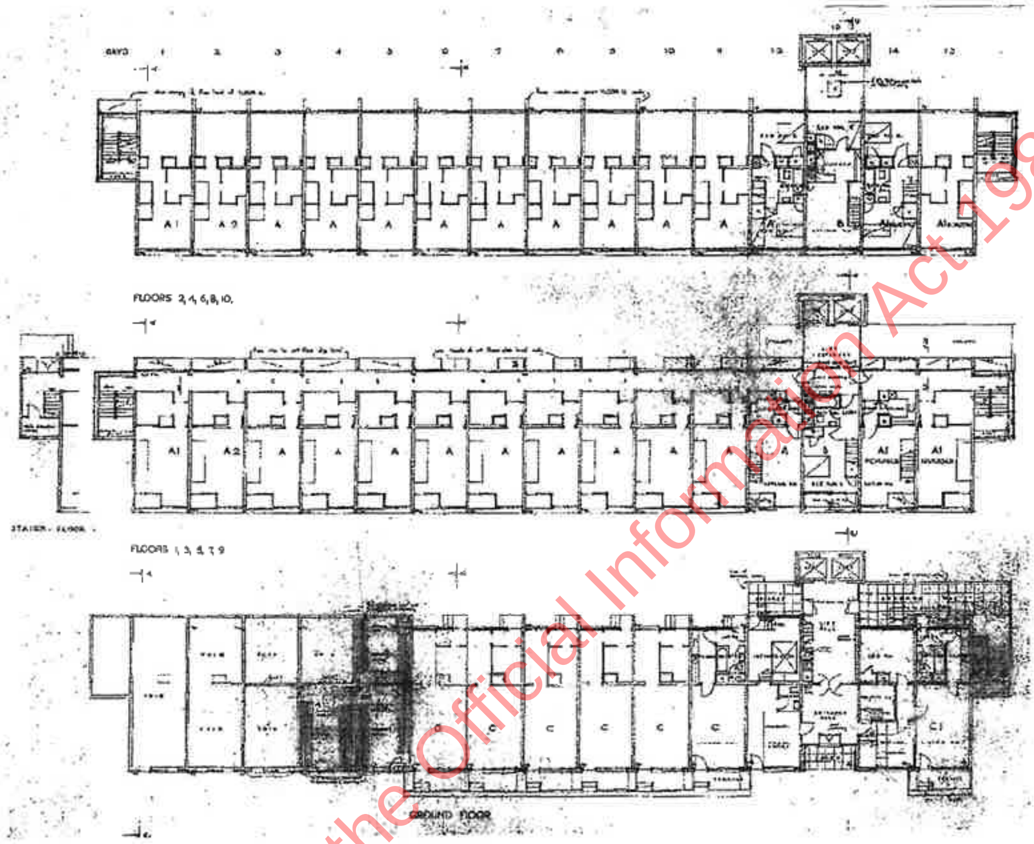
4.1 Photos

East Elevation (front)

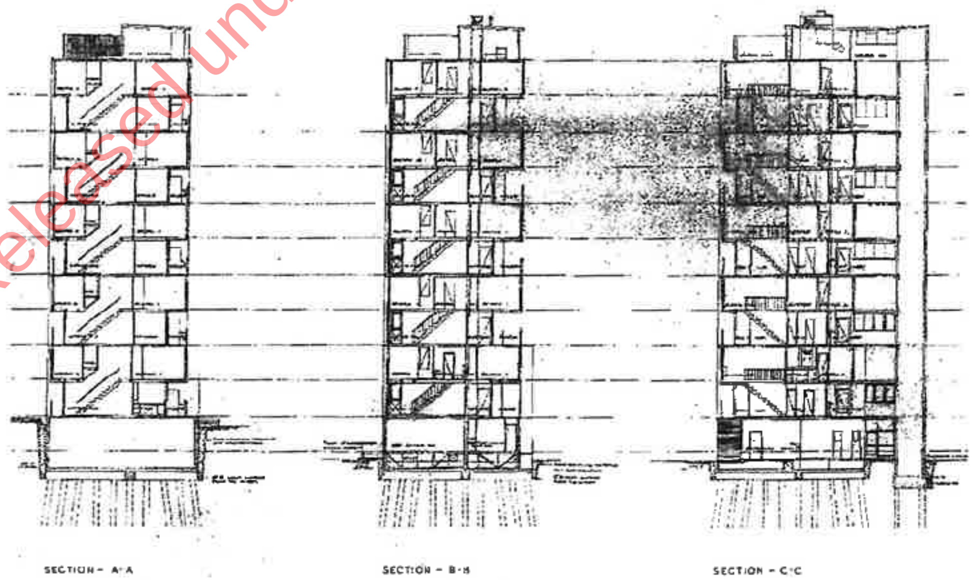
South Elevation

4.2 Drawings

Floor Plans



Cross Sections



4.3 List Relevant Features

A review of the original structural drawings indicates that the building was designed in early 1954 and subsequently constructed making it approximately 55 years old. It was designed by the then Ministry of Works to their codes of practice that were current of the day. At this time the MOW had developed their own codes that were generally more stringent than the corresponding Codes developed by Standards New Zealand and used by the private sector.

The building is 11 stories high with a penthouse running full length over approximately one half of the roof width. The main seismic structural elements are concrete shear walls in each direction. Regularly distributed transverse shear walls with single longitudinal shear wall. The longitudinal shear wall is penetrated by numerous door openings. There are reinforced concrete floors at the sub basement, first, third, fifth, seventh, ninth and roof levels. In between there are timber floors to the ground floor, second floor and even stories thereon up. Foundations are supported on driven concrete piles down to approx 13m max to rock.

4.4 Note Information Sources

	Tick as appropriate
Visual Inspection of Exterior	☑
Visual Inspection of Interior	☒ Partial basesement and typical room
Drawings (note type)	☑ Original
Specification	☒
Geotechnical Reports	☑ Borehole logs on drawings
Other (list)	

5. Determination of $\%NBS_b$

4.1 Determine nominal (%NBS) = $(\%NBS)_{nom}$

a) Date of Design and Seismic Zone

			tick as appropriate
Pre 1935			
1935-1965			✓
1965-1976	Seismic Zone;	A	
		B	
		C	
1976-1992	Seismic Zone;	A	
		B	
		C	
1992-2004			

b) Soil Type

From NZS1170.5:2004, Cl 3.1.3	A or B Rock	
	C Shallow Soil	✓
	D Soft Soil	
	E Very Soft Soil	
From NZS4203:1992, Cl 4.6.2.2 (for 1992 to 2004 only and only if known)	a) Rigid	
	b) Intermediate	

c) Estimate Period, T

	Longitudinal		Transverse	
	≤0.4 seconds		0.50 seconds	
	Fix @ ≤ 0.4sec		concrete shear walls	
	T = ≤ 0.4sec		T = 0.09hn0.75 / Ac0.5	
h_n =		m	31.5	m
A_i =		m^2	26.63	m^2
L_{wi} =		m	8.2	m
A_c =		$\Sigma A_i(0.2 + L_{wi} / h_n)^2$	5.64	$\Sigma A_i(0.2 + L_{wi} / h_n)^2$

Where: h_n = height from the base of the structure to the uppermost seismic weight or mass

A_i = cross sectional shear area of shear wall I in the first storey

L_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces

Restriction: L_{wi} / h_n shall not exceed 0.9

d) $(\%NBS)_{nom}$ determined from Figure 3.3

$(\%NBS)_{nom}$	3.5	**Longitudinal**
$(\%NBS)_{nom}$	4	**Transverse**

Scaling factors:

Note 1: For buildings designed prior to 1965 and known to be designed as public buildings in accordance with the code of the time, multiply$(\%NBS)_{nom}$ by 1.25.
For buildings designed 1965-1976 and known to be designed as public buildings in accordance with the code of the time, multiply $(NBS)_{nom}$ by 1.33 - Zone A
1.2 - Zone B — 1 -

Note 2: For reinforced concrete buildings designed 1976-84 multiply $(NBS)_{nom}$ by 1.2 — 1 -

Note 3: For buildings designed prior to 1935 multiply $(NBS)_{nom}$ by 0.8 except for wellington where the factor may be taken as 1. — 1 -

$(\%NBS)_{nom}$	3.5	**Longitudinal**
$(\%NBS)_{nom}$	4.0	**Transverse**

4.2 Near Fault Scaling Factor, Factor A

If T ≤ 1.5sec, Factor A = 1

a) Near Fault Factor, N(T,D) — 1 Longitudinal; 1 Transverse

(from NZS1170.5:2004, Cl 3.1.6)

b) Near Fault Scaling Factor = 1/N(T,D)

Factor A	1	**Longitudinal**
Factor A	1	**Transverse**

4.3 Hazard Scaling Factor, Factor B

a) Hazard Factor, Z, for site — Z = 0.4

(From NZS1170.5:2004, Table 3.3)

For post 1992 buildings (otherwise leave blank): Z_{1992} =

(Where Z_{1992} is the NZS4203:12992 Zone Factor from accompanying Figure 3.5(b))

b) Hazard Scaling Factor

For pre 1992 = 1/Z — 2.5 Therefore pre 1992

For 1992 onwards = Z_{1992}/Z

Factor B 2.50

4.4 Return Period Scaling Factor, Factor C

a) Building Importance Level — 2

(from NZS1170.0:2004, Table 3.1 and 3.2) — Description: Multi occupancy residential < 5000 people

b) Return Period Scaling Factor from accompanying Table 3.1 — **Factor C** 1

4.5 Ductility Scaling Factor, D

a) Assessed Ductility of Existing Structure, μ

(shall be less than the maximum given in accompanying Table 3.2)

μ =	1.25	Longitudinal
μ =	2	Transverse
max μ =	2	

b) Ductility Scaling Factor

For pre 1976 = k_μ — For 1976 onwards = 1 — Pre 1976

Longitudianal: k_μ = 1.14 — **Factor D** 1.14 **Longitudinal**

Transverse: k_μ = 1.72 — **Factor D** 1.72 **Transverse**

Soil Class= C, Longitudinal Period = 0.4s, Transverse Period = 0.50 s

4.6 Structural Performance Scaling Factor, Factor E

a) Structural Performance Factor, S_p

from accompanying figure 3.4

S_p =	0.925	Longitudinal
S_p =	0.7	Transverse

b) Structural Performance Scaling Factor = $1 / S_p$

Factor E	1.08	**Longitudinal**
Factor E	1.43	**Transverse**

4.7 Baseline %NBS for Building, $(\%NBS)_b$

(equals (%NBS)nom * A * B * C * D * E)

$(\%NBS)_b$	11	**Longitudinal**
$(\%NBS)_b$	25	**Transverse**

6. Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)
A) Longitudinal Direction

Critical Structural Weakness **Effect on Structural Performance** **Building Score**

5.1 Plan Irregularity

Effect on Structural Performance	Severe	Significant	Insignificant		
	0.4 max	0.7	1	**Factor A**	1

Comment: ---

5.2 Vertical Irregularity

Effect on Structural Performance	Severe	Significant	Insignificant		
	0.4 max	0.7	1	**Factor B**	1

Comment: ---

5.3 Short Columns

Effect on Structural Performance	Severe	Significant	Insignificant		
	0.4 max	0.7	1	**Factor C**	1

Comment: ---

5.4 Pounding Potential
(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect
Select appropriate value from table
Note:
Values given assume the building has a frame structure. For stiff buildings (e.g. with shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1	Severe	Significant	Insignificant
Seperation	0<Sep<.005H	.005<Sep<0.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	0.7	0.8	1
Alignment of Floors not within 20% of Storey Height	0.4	0.7	0.8

a) Factor D2: - Height Difference Effect
Select appropriate value from table

Factor D2 1

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<0.01H	Sep>.01H
Height Difference > 4 Storeys	0.4	0.7	1
Height Difference 2 to 4 Storeys	0.7	0.9	1
Height Difference < 2 Storeys	1	1	1

(Set D = Lesser of D1 and D2 or...
set D = 1.0 if no prospect of pounding)

☑ Check if no prospect of pounding

Factor D 1

5.5 Site Characteristics - (Stability, landslide threat, liquefaction etc.)

Effect on Structural Performance	Severe	Significant	Insignificant		
	0.5 max	0.7	1	**Factor E**	1

Comment: ---

5.6 Other Factors
For ≤ 3 Storeys - Maximum value 2.5
otherwise - Maximum value 1.5. No minimum

Factor F 1.5

Rationale for choice of Factor F:
Squat shear wall with concrete diaphragm, many regular distributed transverse shear walls

5.7 Performance Achievement Ratio (PAR)
(equals **A * B * C * D * E * F**)

Longitudinal (PAR) 1.5

B) Transverse Direction

Critical Structural Weakness	**Effect on Structural Performance**				**Building Score**
5.1 Plan Irregularity					
Effect on Structural Performance	Severe 0.4 max	Significant 0.7	Insignificant 1	**Factor A**	1
Comment: ---					
5.2 Vertical Irregularity					
Effect on Structural Performance	Severe 0.4 max	Significant 0.7	Insignificant 1	**Factor B**	1
Comment: ---					
5.3 Short Columns					
Effect on Structural Performance	Severe 0.4 max	Significant 0.7	Insignificant 1	**Factor C**	1
Comment: ---					

5.4 Pounding Potential
(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect
Select appropriate value from table
Note:
Values given assume the building has a frame structure. For stiff buildings (e.g. with shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 1

Table for Selection of Factor D1

Seperation	Severe 0<Sep<.005H	Significant .005<Sep<0.01H	Insignificant Sep>.01H
Alignment of Floors within 20% of Storey Height	0.7	0.8	1
Alignment of Floors not within 20% of Storey Height	0.4	0.7	0.8

a) Factor D2: - Height Difference Effect
Select appropriate value from table

Factor D2 1

Table for Selection of Factor D2

	Severe 0<Sep<.005H	Significant .005<Sep<0.01H	Insignificant Sep>.01H
Height Difference > 4 Storeys	0.4	0.7	1
Height Difference 2 to 4 Storeys	0.7	0.9	1
Height Difference < 2 Storeys	1	1	1

(Set D = Lesser of D1 and D2 or...
set D = 1.0 if no prospect of pounding)

☑ Check if no prospect of pounding

Factor D 1

5.5 Site Characteristics - (Stability, landslide threat, liquefaction etc.)

Effect on Structural Performance	Severe 0.5 max	Significant 0.7	Insignificant 1	**Factor E**	1
Comment: ---					

5.6 Other Factors
For ≤ 3 Storeys - Maximum value 2.5
otherwise - Maximum value 1.5. No minimum

Factor F 1.5

Rationale for choice of Factor F:
Many regular distributed transverse shear walls, concrete diaphragm.

5.7 Performance Achievement Ratio (PAR)
(equals **A * B * C * D * E * F**)

Transverse (PAR) 1.5

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7. Percentage of New Building Standard (%NBS)

		Longitudinal	Transverse
6.1 Assessed Baseline (%NBS)$_b$ (from Table IEP-2)		11	25
6.2 Performance Achievement Ratio (PAR) (from Table IEP-3)		1.50	1.50
6.3 PAR x Baseline (%NBS)$_b$		16	37
6.4 Percentage New Building Standard (%NBS) (use lower of two values from step 4.3)			16
6.5 Potentially Earthquake Prone? (Mark as appropriate)	%NBS ≤ 33		Yes
6.6 Potentially Earthquake Risk? (Mark as appropriate)	%NBS ≤ 67		Yes
6.7 Provisional Grading for Seismic Risk based on IEP	Seismic Grade		E

Evaluation prepared by:		**Approved By:**	
Name	**P Johnson**		**R N Patton**
CPEng. No	-		**026288**

Relationship between Seismic Grade and %NBS:

Grade:	A+	A	B	C	D	E
%NBS:	>100	100 to 80	80 to 67	67 to 33	33 to 20	<20

8. Appendix

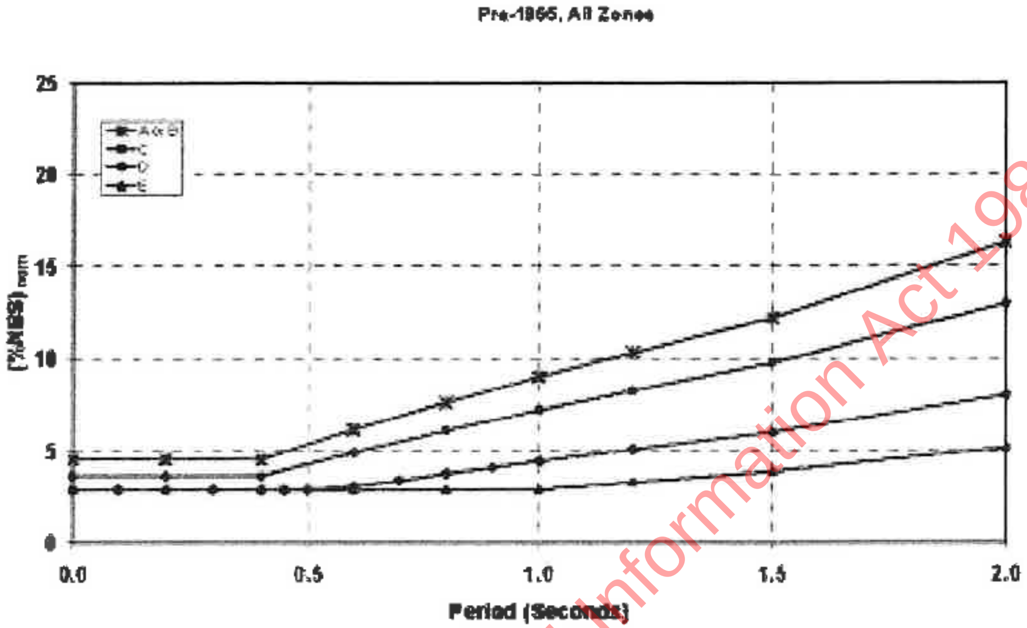
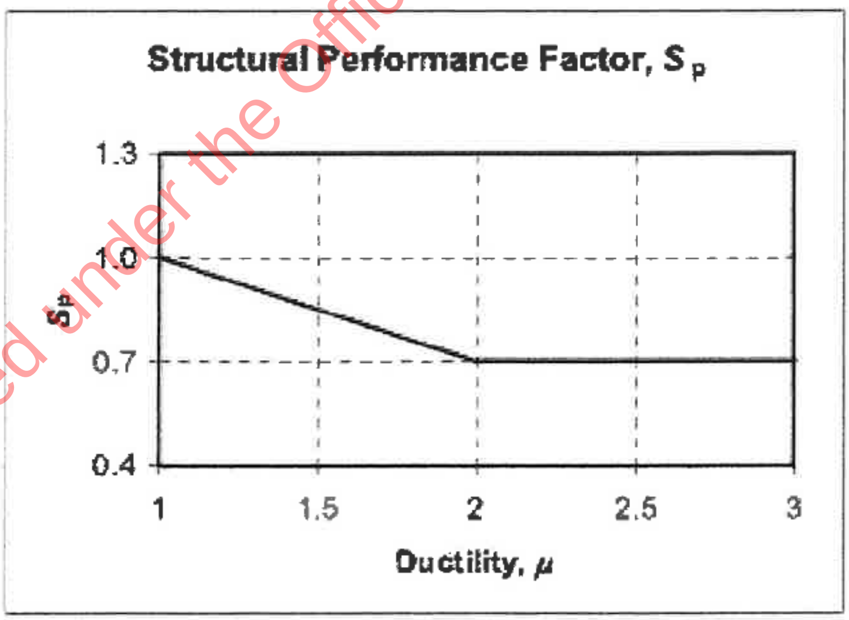


Figure 3.3(a): (%NBS)$_{nom}$ Pre- 1965, All Zones



the Structural Performance Factor from NZS1170.5:20

Figure 3.4: Structural performance factor, S_p

Figure 3.5(a) NZS 4203:1984,
NZS 4203: 1976 and
NZS 1900:Chapter 8 sim.

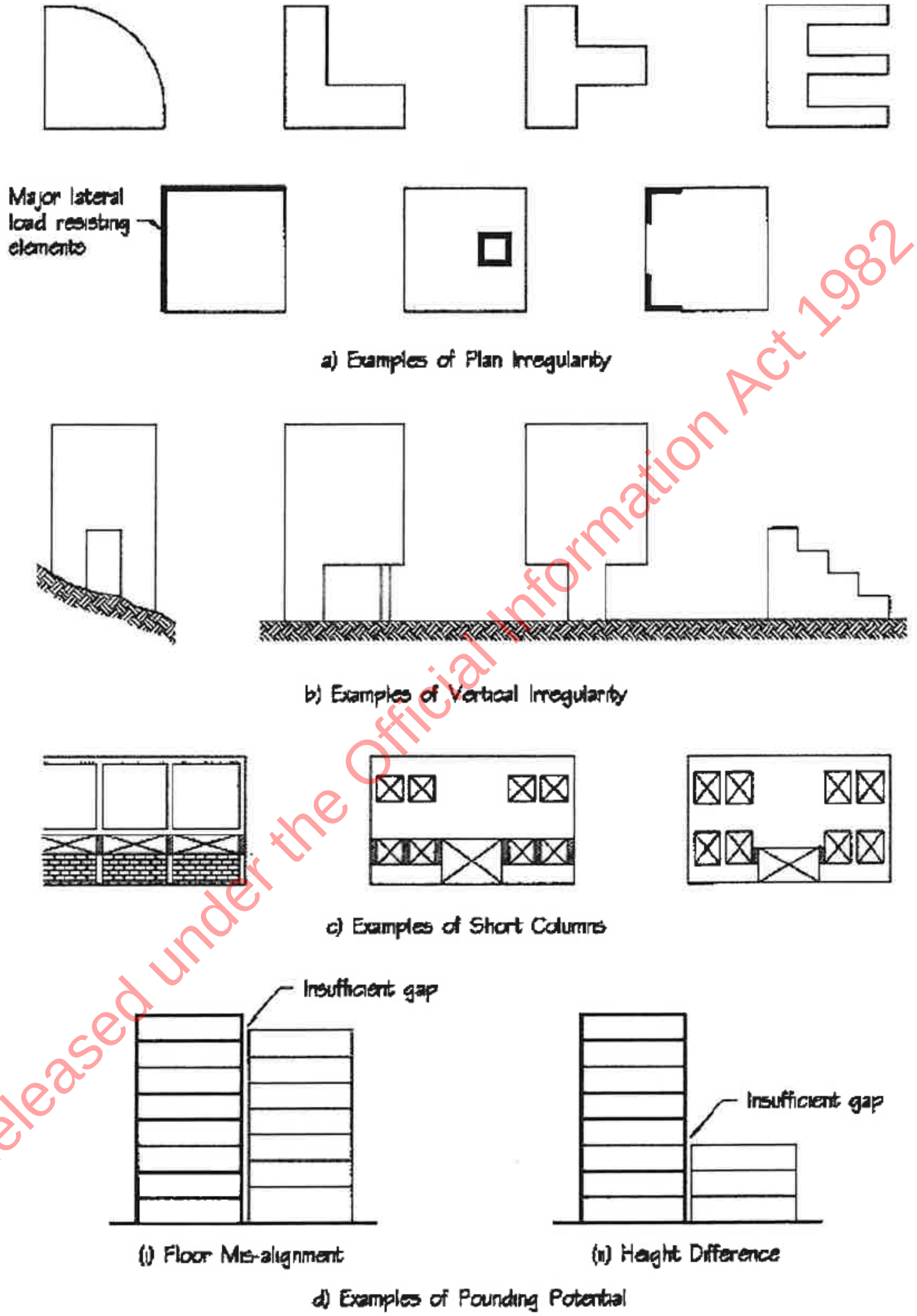


Figure 3.7: Examples of critical structural weaknesses

Table 3.1: Return period scaling factor

NZS1170.5:2004 Return Period Factor, *R*				Return Period Scaling Factor, *C*			
Importance level	Comment	Annual Probability of Exceedance	Return Period Factor, *R*	Pre 1965	1965-76	1976-92	1992-04
1	Minor structures (failure not likely to endanger human life)	1/100	0.5	2	2	2	1.2
2	Normal structures and structures not falling into other levels	1/500	1	1	1	1	1
3	Major structures (affecting crowds)	1/1000	1.3	0.8	0.8	1.1	0.9
4	Post-disaster structures (post-disaster functions or dangerous activities)	1/2500	1.8	0.6	0.6	1	0.7
5	Exceptional structures are outside the scope of the IEP, special study required.						

Where R is the return period factor appropriate to the current use of the building, as shown in Table 3.5 of NZS 1170.0:2002

Table 3.2: Ductility factors to be used for existing buildings

Structure Type	Maximum allowable ductility factor for IEP			
	Pre-1935	1935-65	1965-76	1976-2004
All buildings	2	2	2	6

Table 3.3: Ductility scaling factor

	Structural Ductility Scaling Factor, k_μ							
	1.0 or less		1.25		1.50		2	
Soil Type	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E
Period, *T*								
≤ 0.40s	1	1	1.14	1.25	1.29	1.50	1.57	1.70
0.50s	1	1	1.18	1.25	1.36	1.50	1.71	1.75
0.60s	1	1	1.21	1.25	1.43	1.50	1.86	1.80
0.70s	1	1	1.25	1.25	1.50	1.50	2.00	1.85
0.80s	1	1	1.25	1.25	1.50	1.50	2.00	1.90
≥1.00s	1	1	1.25	1.25	1.50	1.50	2.00	2.00

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Table 3.4: Guide to severity of critical structural weaknesses

	Effect on structural performance		
Critical structural weakness	**Severe**	**Significant**	**Insignificant**
Plan irregularity			
L-shape, T-shape, E-shape	Two or more wings length/ width > 3.0, or one wing length/width >4	One wing length/width > 3.0	All wings length/width ≤ 3.0
Long narrow building where spacing of lateral load resisting elements is ...	> 4 times bldg. Width	> 2 times bldg. Width	≤ 4.0 times bldg width
Torsion (Corner Building)	Mass/centre of rigidity offset > 0.5 width	Mass/centre of rigidity offset > 0.3 width	Mass/centre of rigidity offset ≤ 0.2 width or effective torsional resistance available from elements orientated perpendicularly.
Ramps, stairs, walls, stiff partitions	Clearly grouped, clearly an influence	Apparent collective influence	No or slight influence
Vertical irregularity			
Soft storey	Lateral stiffness varies > 150%	Lateral stiffness varies 100–150%	Lateral stiffness varies < 100%
Mass variation (geometrical)	Mass varies > 150% between adjacent floors	Mass varies 100–150% between adjacent floors	Mass varies < 100% between adjacent floors
Vertical discontinuity	Any element contributing > 0.5 stiffness of the lateral force resisting system discontinues vertically	Any element contributing > 0.3 stiffness of the lateral force resisting system discontinues vertically	Elements contributing to the lateral force resisting systems are continuous vertically
Short columns			
Columns < 70% storey height between floors clear of confining infill, beams or spandrels	Either > 80% short columns in any one side Or > 80% short columns in any storey	> 60% short columns in adjacent sides > 60% columns in a storey are short	No, or only isolated, short columns
Pounding effect			
Floor aligns ≤ 20% storey height	$0 <$ separation $< 0.005\ H$	$0.005\ H <$ separation $< 0.01\ H$	Separation $> 0.01\ H$
Floor aligns > 20% storey height	$0 <$ separation $< 0.005\ H$	$0.005\ H <$ separation $< 0.01\ H$	Separation $> 0.01\ H$
	where H = height to the level of the floor being considered		
Height difference effect			
No adjacent building, or height difference < 2 storeys	$0 <$ separation $< 0.005\ H$	$0.005\ H <$ separation $< 0.01\ H$	Separation $> 0.01\ H$
Height difference 2–4 storeys	$0 <$ separation $< 0.005\ H$	$0.005\ H <$ separation $< 0.01\ H$	Separation $> 0.01\ H$
Height difference > 4 storeys	$0 <$ separation $< 0.005\ H$	$0.005\ H <$ separation $< 0.01\ H$	Separation $> 0.01\ H$
	where H = height of the lower building and separation is measured at H		
Site characteristics	Unstable site Extensive landslide from above Probable liquefaction	Potential for site instability Landslide from above Liquefaction potential	Not a significant threat