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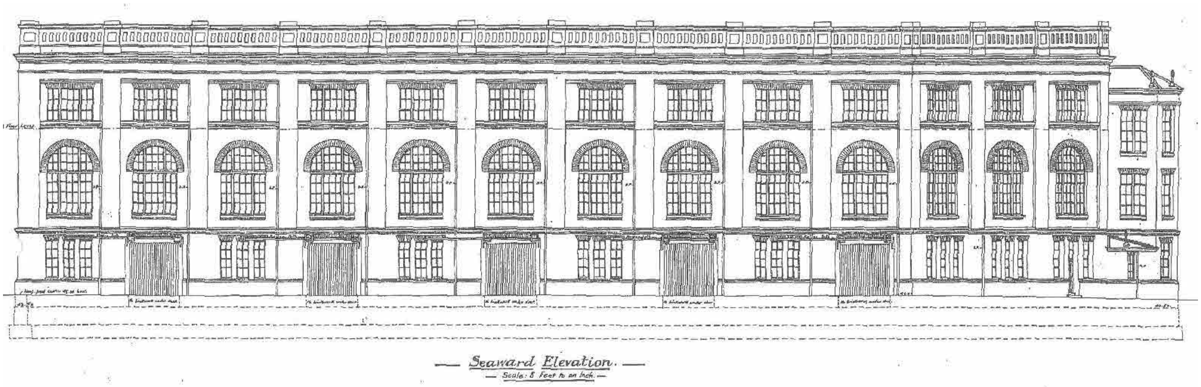

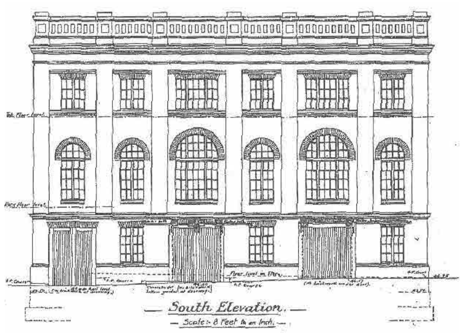

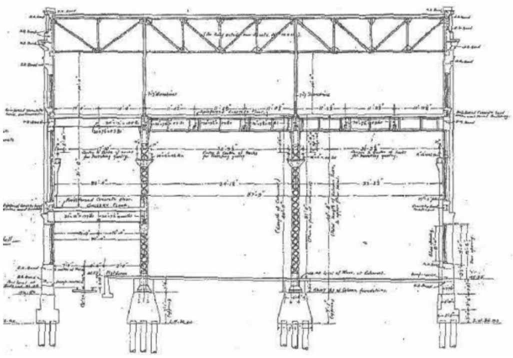

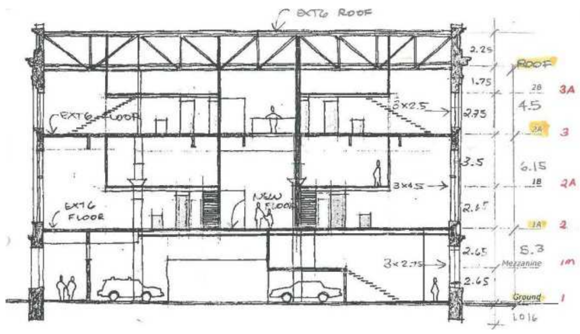

Detailed Seismic Assessment of the Waterloo Apartments Building – 28 Waterloo Quay - Wellington

Report ref:
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Contents

1. Executive Summary – Detailed Seismic Evaluation of the Waterloo Apartments

1.1 Seismic Assessment

This study assesses the seismic capacity of the retro-fitted Waterloo Apartments building using the current seismic design code. AS/NZS 1170.5:2004. This code is referred to as the “New Building Standard” (NBS).

The assessment is based on the assessment processes in the New Zealand Society for Earthquake Engineering (NZSEE) publication “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes”; this is the quasi-standard used for seismic assessment in New Zealand. It is colloquially known as the “Red Book”.

This assessment concludes that the building structure has a New Building Standard value of 69% based on the lateral strength capacity of the building compared to the demand placed on it by a current code earthquake.

Note that, at 69%, the building is quite typical for other buildings that were upgraded or retrofitted in the early 2000s. It is robust due to the reinforced concrete shear walls on the interior apartment wall locations and due, in part, to its original structural steel construction.

The capacity of the foundation system could be less than 69%NBS due to the possibility of the lateral slumping of the adjacent sea wall being caused by an earthquake shaking the ground with less than 69% NBS equivalent levels of peak ground acceleration (PGA).

Refer to the following section 1.2.

In section 10 - The New Zealand Society for Earthquake Engineering Grading Scheme – there are two tables and descriptions of the grading scheme. At 69% NBS, the building has a “B Grade” rating from NZSEE table 2.1 and is a Low Risk in terms of table 2.2.

It should be noted that the building came through the Sunday 21st July Cook Strait earthquake and its lesser aftershocks with no significant structural damage. That report is attached as Appendix G – Comparative Size of the July 2013 Cook Strait Earthquake.

1.2 Geotechnical Issues

The building was constructed on reclaimed land in 1909 and the reclamation was placed behind a mass concrete sea wall at some time in the late 1980’s.

Refer to Figure 1 - Cross section through the foundations and sea wall.This is an extract from the Tonkin and Taylor report.

The building is supported on closely spaced driven timber piles capped with a substantial ring beam / pile cap. At the stage when redevelopment was being considered, the piles were checked and found to be in good order. Refer to Section 12 - Appendix B – BECA Report on Existing Piles and to Section 13 - Appendix C – BECA Timber Pile Inspection.

Irrespective of the piling, the block of land between the building and the sea wall could migrate sideways (eastwards towards the sea) during a large earthquake as the sea wall slumps to the east under earthquake loading..

This slumping of the sea wall could cause the ground between Waterloo Quay and the sea wall to progressively migrate (or stretch) towards the seaward side. Current analysis by geotechnical engineers, Tonkin and Taylor, estimate that some movement could occur at PGA levels equivalent to 50% NBS and increase in magnitude to a degree that could cause damage to the building at 75% NBS levels of ground shaking.

There is some degree of latitude in assessing the onset of potentially damaging lateral movement as geo-mechanics is an inexact science, Therefore Tonkin and Taylor have given a range of potential estimated movements.

Based on the range of movement estimated by Tonkin and Taylor in their report we, as structural engineers, consider that the building should be able to tolerate the ground movements that could be expected to result from between 50% and 60% NBS shaking. To reflect the inexact nature of the science of geotechnical analysis, we settle on 55% NBS for the purposes of this report.

If work is done to strengthen the resistance of the block of land moving seawards then the capacity of the subsoil and the foundation system can be increased up to the same 69% NBS that the building has been assessed at.

This issue is addressed in detail in the geotechnical report appended – refer to section 16 - Appendix F – Geotechnical Seismic Stability of the Site.

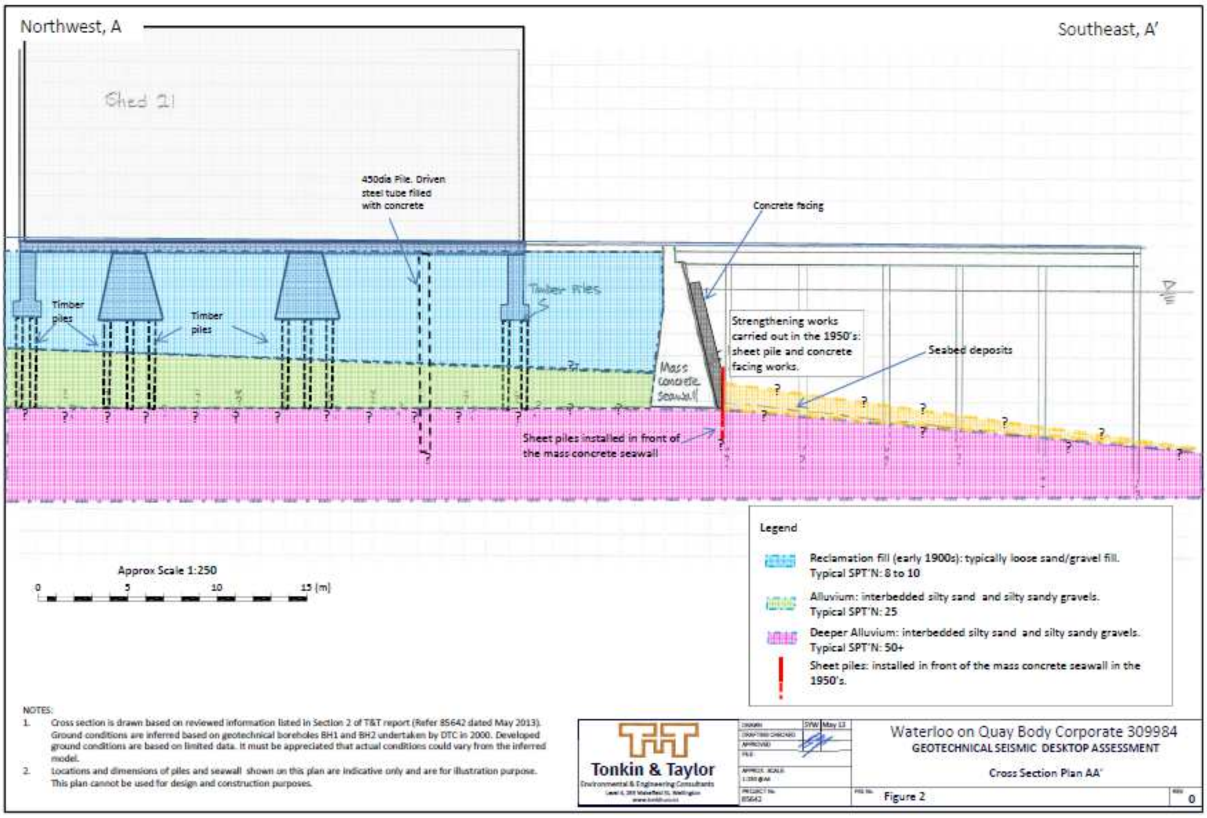


Figure 1 - Cross section through the foundations and sea wall

2. Limitation

This report is not a condition report on the building. It is purely a Seismic Assessment. It does not cover any other building issues apart from *Structure – B2* in the Building Code. Refer to section 15 - Appendix E – Synopsis of the Building Code.

3. Building – General Description

The building was originally built as a wool-store in about 1909. It was a strong and very good quality building at its conception.

The apartment conversion was designed by Athfield Architects and structural engineers Dunning Thornton Ltd circa 2000. They respected the existing structure to a large degree. The major changes were:

- Foundation enhancement
- Reinforced concrete and reinforced inter-tenancy block-work walls that are also structural shear walls
- Additional floors for apartments, both concrete and timber
- Stabilising secondary structural steel to the brickwork walls
- New lifts and stairs at each end of the building

The plan below shows that the building has new concrete shear walls running both longitudinally (North / South) and transversely (East / West).

The concrete walls are shown in yellow and the major structures are on grids 2 (B to C, D to E, I to J, K to L) longitudinally and grids E,G and I transversely. Shear walls are a very robust seismic system and are coupled to an upgraded foundation system capable of resisting the shear and overturning forces.

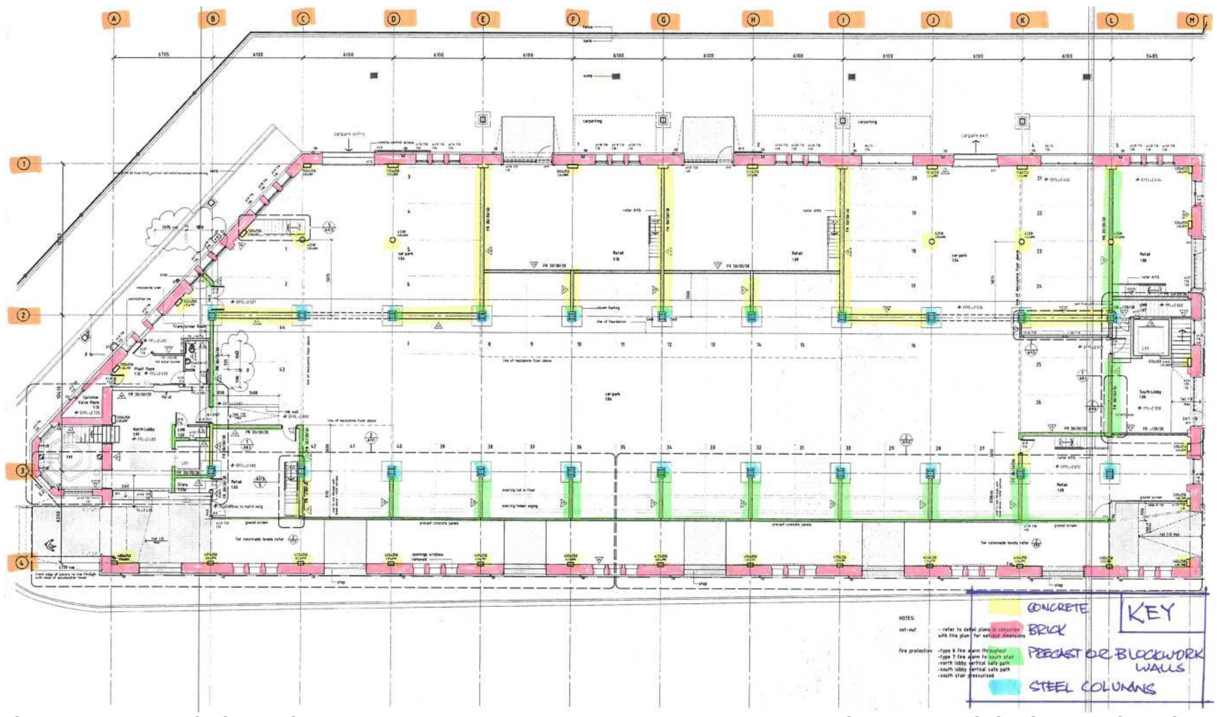
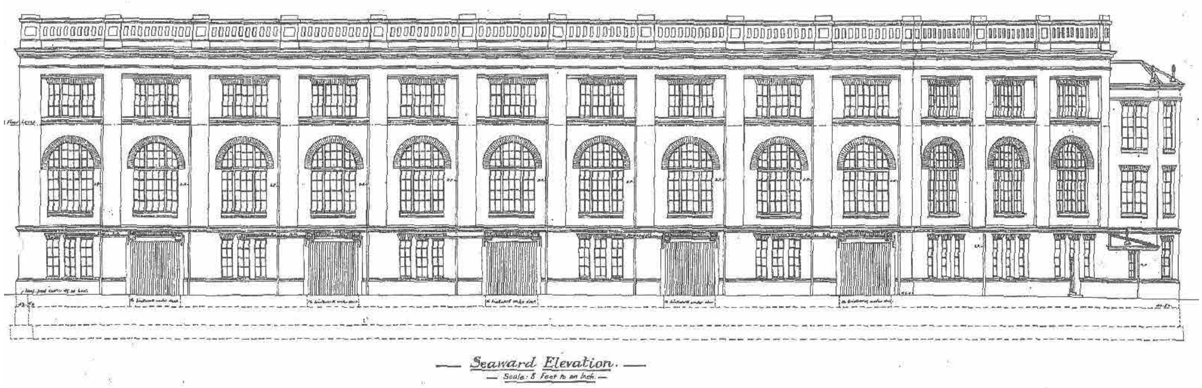


Figure 2 - Ground Floor Plan

Figure 3 - Original East Elevation



4. Assessment Methodology - General Approach to Assist in Understanding this Report

The general approach to assess the seismic capacity of existing buildings is simple. This logic should be used to assess all older buildings. It has four distinct and initially unrelated steps:

- Understand the configuration of the building, its materials and construction details and calculate its seismic mass. Understand its environs – step 1
- Determine the lateral load **capacity** of the building – step 2.
- Determine the code seismic **demand** (lateral loads) on the building – step 3.
- Divide the lateral load **capacity** by the code seismic **demand** to obtain %NBS – step 4.

The detail and rigour in these four steps need to match the level of complexity of the building and its fabric together with any site specific issues

4.1 Assessment Methodology - Step 1 - Project Data Gathering

4.1.1 General comments

Collate and register all available documentation on the building regardless of the discipline. Include drawings, specifications, calculations and older photos.

Collate and issue this information. Provide a summary of the information and any relevant interpretation of old data that may need clarification.

Collate and register all available documentation on geotech. Provide a summary of the information and relevant interpretation.

Calculate the component weights for each building on site. Assemble into a spread-sheet with percentages and totals.

4.1.2 Geotech Input

Prepare and implement a Geotech Investigation with an accent on soil dynamics, soil structure interaction, lateral spreading and potential liquefaction.

Obtain a foundation condition / assessment including likely damping values. Damping is akin to shock absorbers in a car's suspension; damping absorbs energy and lessens seismic loads.

4.1.3 Design Features Report

We need to include (at least) these points in a Design Features Report as a commentary for old buildings. They are:

- A clear description of the load paths assumed in this structure, both lateral and gravity
- A clear description of the resisting elements attached to the same load paths.
- A clear description of the structural elements, or quasi structural elements, which are not considered capable of resisting lateral loads. The reasons for not using these should be stated and, if necessary, reference made to future testing as below.
- Testing or data gathering on potential structural element strengths/capacities. This is very necessary to make sure that the maximum resistances are obtained for the building. If we do not know the strength of things, we should not just ignore them. Some allowance must be made.
- Damping. The above comments also apply to any damping that may occur in the structural system.

- Foundations systems. Soil / Structure interactions are important; not least the rocking of the building on the soil. If the building is not piled, that should be taken into account.
- Estimation of the period of the building – the time it takes to swing backwards the forwards through on cycle of shaking. This is a crucial parameter because it sets the overall seismic demand on the building. It is possible that, with new technology out of Canterbury University, a seismograph may be used to work out the first mode periods. This will give conservative results.
- Vulnerability and critical structural weakness issues.
- Any parts and portions that may lead to risk and / or collapse.

4.2 Assessment Methodology - Step 2 - Determine the Lateral Capacity of the Building

Assess the lateral load capacity of the building, using the best of the capacity of all the components. The capacity might be a strength capacity or, alternatively, a capacity to add damping or to afford ductility. Ductility is a measure of how the building structure performs when it is overstressed. A “ductile” building is quite forgiving when over-stressed. It “gives and bends” rather than “snapping” like a brittle twig. This is covered in the NZSEE publication “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes”;

One must look at the lateral load **capacity** of the building in a holistic way without initially confusing it with the earthquake lateral load **demand**.

A building has a “capacity” to resist lateral loads.

An earthquake imposes lateral load “demands” on the building

Therefore the seismic lateral load “demands” on the building must be resisted by the building’s “capacity” to resist those lateral loads.

The calculated capacity depends on how well one models and analyses the building. It will depend on how strong the materials are. (The assessing engineer may advocate some testing to determine strengths.) It will depend on any areas of critical weakness and how they may be partially mollified by re-distribution of loads or subsequently improved by physical strengthening.

The key is to understand the building and its environs and analyse it very well in order to realise as much lateral capacity as possible.

Analysis methods should always begin with a “hand” study. As engineers, we are not designing a new building: we are analysing the capacity of an existing building with known member sizes, capacities and with some ability to transfer loads (re-distribution) when overstressed.

Although not necessary for this particular building, due to its relatively simple structure and squat structure, computer studies can be carried out. These include, in increasing levels of providing more knowledge, elastic analyses (ETabs) followed by Non Linear Modal and Time History Analysis (SAP 2000). These will increase the detail of a simple “push-over” analysis to better assess any post-elastic performance of the building.

Materials and their properties can be assessed by:

- Using current design codes where appropriate
- Testing - either component materials or full scale in-situ
  Using the NZSEE publication “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes”. (Note - This has been the main plank of this study)
- Using the American Society for Civil Engineering (ASCE) publication - “Seismic Rehabilitation of Existing Buildings” - ASCE/SEI 41-06
- Using the Applied Technology Council (ATC) - “Seismic Evaluation and Retrofit of Concrete Buildings” – ATC40

- Using judgement and Interpretation. It is imperative that the assessor lists what they have NOT considered due to lack of information.

Generally, the lateral load capacity of a building can be **enhanced** with greater engineering effort and understanding.

4.3 Assessment Methodology - Step 3 - Determine the Code Design Level Demand

The aim of this section is to determine, as accurately as possible, a realistic (not conservative) “New Building Standard” seismic coefficient. A number of different code defined parameters are used to establish the lateral **demand** on the building when multiplied by the building’s weight.

The code acceleration depends on a number of design parameters that include:

- Period
- Ductility
- Zone (the seismicity of the region within New Zealand)
- Subsoil Classification
- Importance level – Larger more “important” buildings are designed to resist higher load levels.
- Damping
- Soil structure interaction

The ductility, the period, the soil type and the damping all have an element of variability and judgement.

The charts below attempt to show the sensitivity to the **demand** on a building. For example, soil classification Class C has been used as a benchmark below. Use Class D and it can increase the loads by 30% to 60% depending on the period of the building. Equally, a ductility factor of 4 gives a demand of only 30% of that based on a ductility factor of 1. The assessment of the period is crucial. Soil / structure interaction can add lots of beneficial damping.

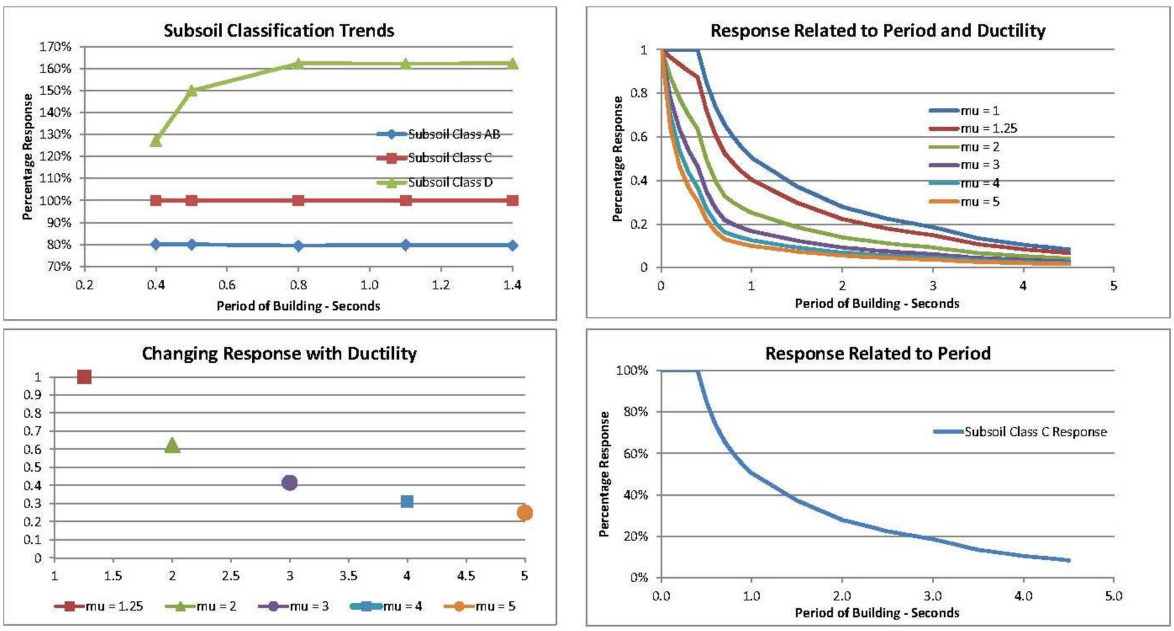


Figure 4 - Charts showing the sensitivity of the selection of code design parameters

Where buildings have neighbours, we need to assess any potential pounding issues between the building and the adjoining buildings. Pounding (banging together) is not necessarily a large issue in reinforced concrete and structural steel buildings.

The code acceleration also depends on the importance level selected. Normally Importance Level 2 (IL2) is selected which gives a risk factor of 1.0.

Note that the selection of these parameters, which culminate in assessing the lateral load **demand** is quite independent from establishing the lateral load **capacity** of the structure. The exception is that the lateral load capacity of the building will provide some insight into the ductility of the structure.

Generally, the seismic demand on a building can be reduced with greater engineering effort and understanding.

4.4 Assessment Methodology - Step 4 - Determine % NBS

The %NBS figure is simply the lateral load **capacity** of the building divided by the lateral load **demand** on the building.

The %NBS figure can fluctuate wildly if either the demand or the capacity (or both) are wrongly assessed.

The %NBS value can increase with increasing assessment effort to:

- Increase the CAPACITY
- Lower the DEMAND

5. Step 1 - Understanding the Building and its Environs

5.1 Building configuration

A relatively good archival record was held by Wellington Waterfront Ltd.

Drawings of some of the plans, elevations and sections from both the archives and from the year 2000 alterations follow:

5.2 Foundations

The detail following is mainly for background information.

5.2.1 Geotechnical Considerations – Refer to Section 11 - Appendix A - Geotechnical Data Circa 2000

These are the bore-logs, CPT results and position of the bores

5.2.2 Original Foundation System – Refer to Section 12 - Appendix B – BECA Report on Existing Piles

The position and condition of the old existing timber piles was a critical issue at the time of re-construction. This particular report clarifies those issues

5.2.3 Original Timber Piles – Refer to Section 13 - Appendix C – BECA Timber Pile Inspection

As above, the position and condition of the old existing timber piles was a critical issue at the time of re-construction. This particular report clarifies more issues.

5.3 Pounding Issues

The building stands alone from its neighbours. There are no pounding issues where neighbouring building can “clap hands” during an earthquake and cause damage to each other.

5.4 Design Features

These are some relevant issues:

- The structural elements of the building’s retro-fit were designed by structural engineers Dunning Thornton Ltd, a well-respected Wellington consultant. Their calculations were made available to the author and the subsequent review found them to be detailed and of quality reasoning. They did carry out a modal analysis of the building using a well-known software package called ETabs. The drawings are clear and well detailed.

- The gravity load paths are simple; the one-way-spanning floor slabs are supported on primary beams or concrete (block or in-situ) walls.

- The original steel frames (beams and columns) have not been relied upon to resist lateral loads. Because the building has extremely stiff walls compared to the frames; the walls will resist the majority of the lateral loads. The frames alone contribute relatively little seismic load resistance.

- The Dunning Thornton year 2000 design did not take the brick-work façade into account because it is brittle – constructed from un-reinforced brick masonry. Dunning Thornton used a ductility factor of 3.0 in their overall design.

  The NZSEE “Red Book” allows a ductility factor of 1.5 to be used when assessing brick-work. Consequently, as a further check, a separate seismic assessment was carried out using a DEMAND based on $\mu = 1.5$. However, this approach, as expected, resulted in a %NBS lower than 69%. This is because the much higher lateral load demand resulting from a low ductility overshadowed the considerable increase in lateral load capacity provided by the brick walls in shear.

- No testing was needed. Concrete and steel strengths were taken as recommended by the “Red Book” or from the Dunning Thornton specification.

- No foundation damping was considered and therefore no benefit was derived from any soil / structure interactions.

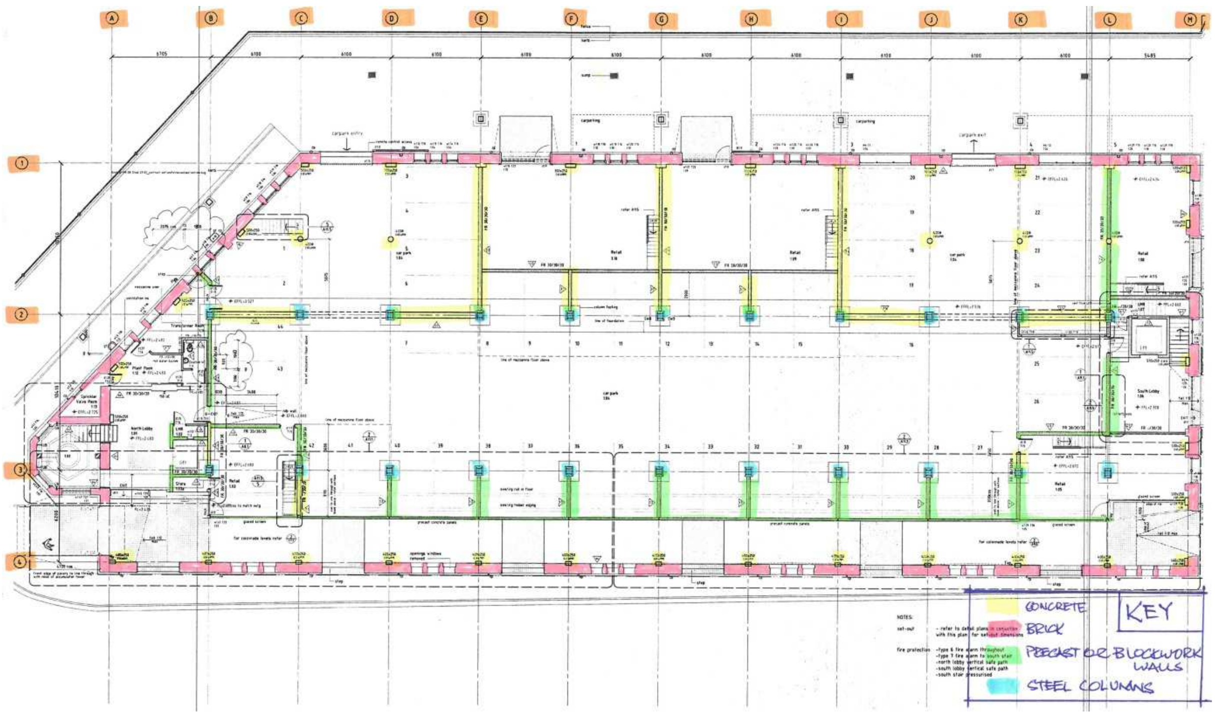
- Estimation of the period – this was taken at less than 0.4 seconds; the top of the response spectra. It is covered in the Dunning Thornton calculations.

- The building has no significant vulnerability or critical structural weakness issues in terms of the structural system. The unreinforced brickwork may suffer damage in a 1:500 year earthquake but building collapse is unlikely due to the reinforced concrete elements and the structural steel frame and wall mullions.

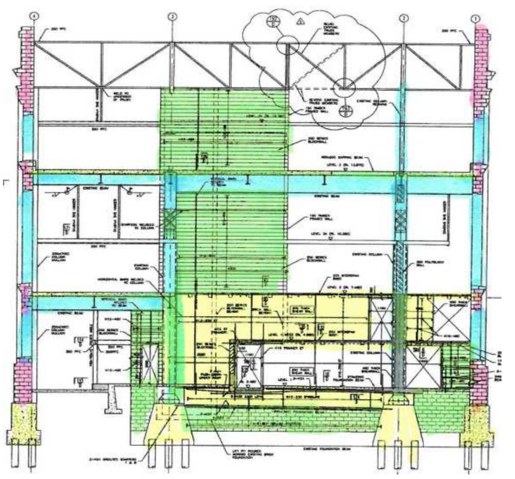
- Equally, it does not have any "parts and portions" that may lead to risk and / or collapse. The floors are very good diaphragms and will distribute any local loads easily. The parapets have been strengthened.

5.5 Shear Wall Configuration

The following are a series of cross-sections through the building that outline the shear wall system constructed in the early 2000's that form the bulk of the seismic strengthening system.
The plan reproduced from above, shows the grid and wall locations.

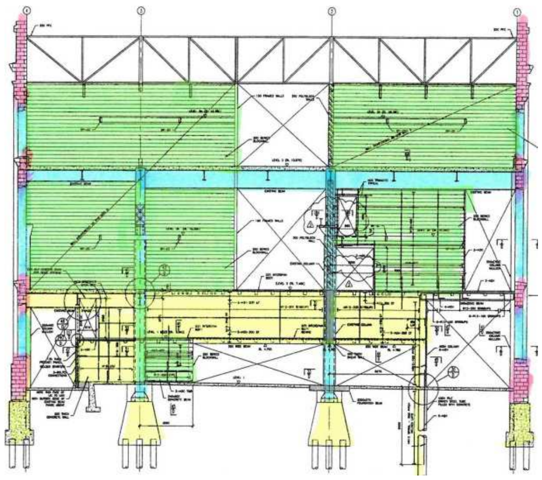


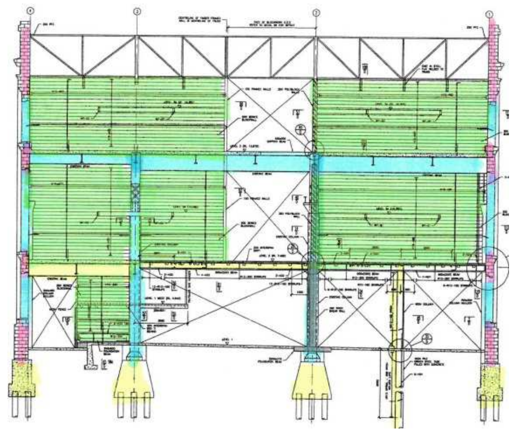
These cross-sections are on each grid-line and are part of the transverse seismic resistance system. They are presented to indicate the considerable scope of the walls that resist seismic loads in the building



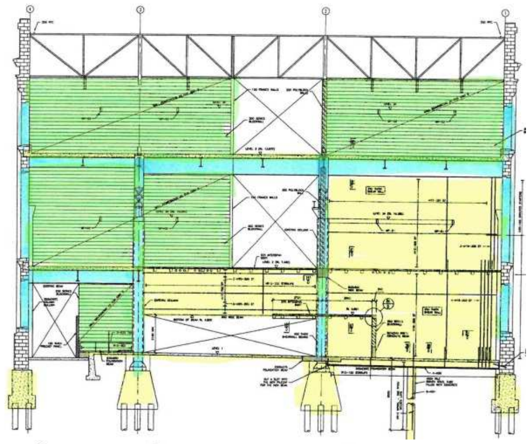
Grids

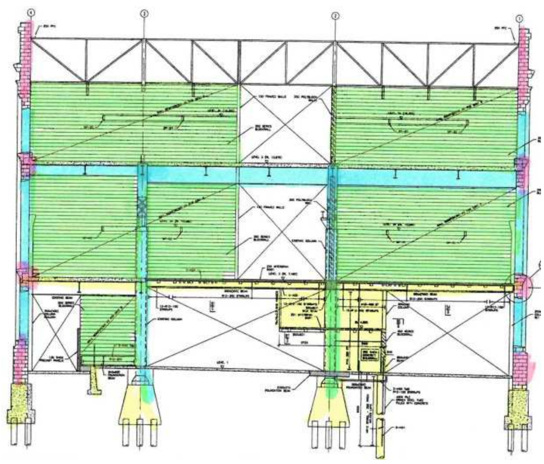
B - C





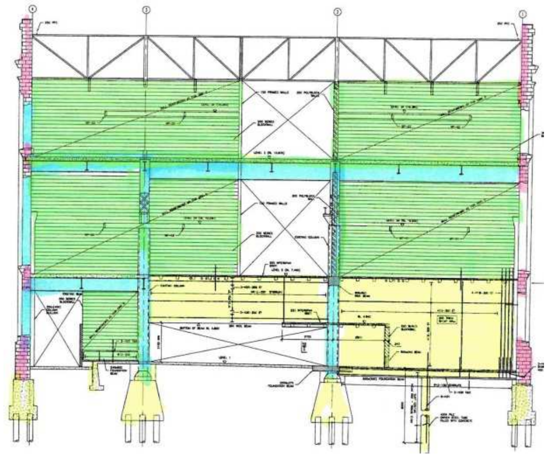
Grids
D - E

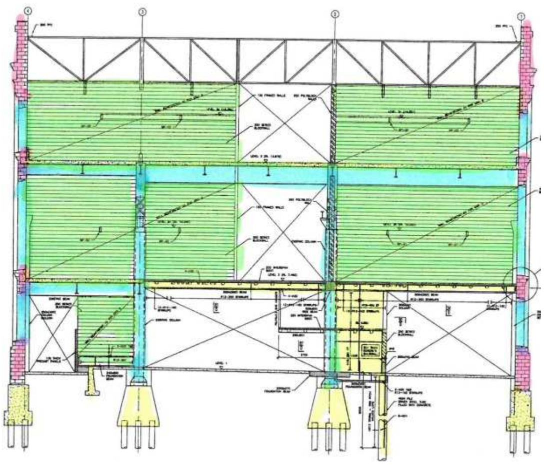




Grids

F- G





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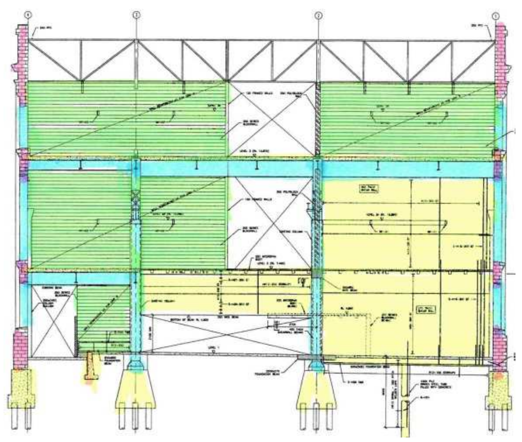
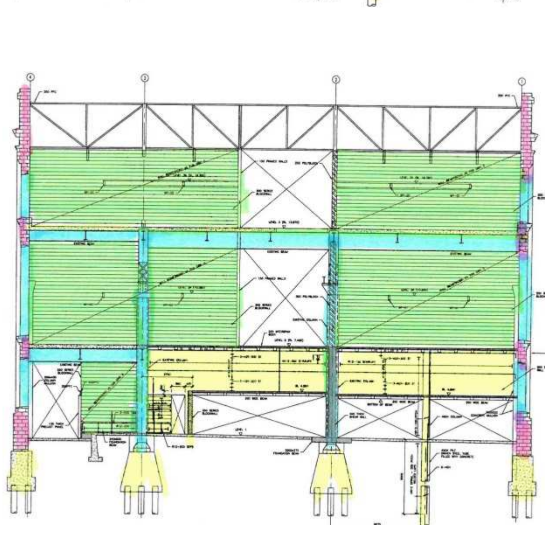
H - I





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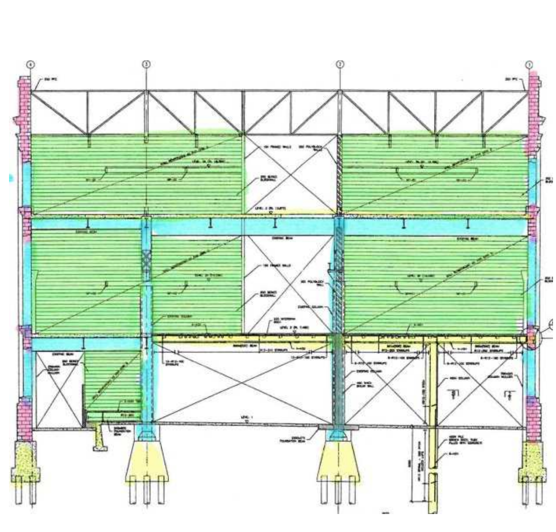
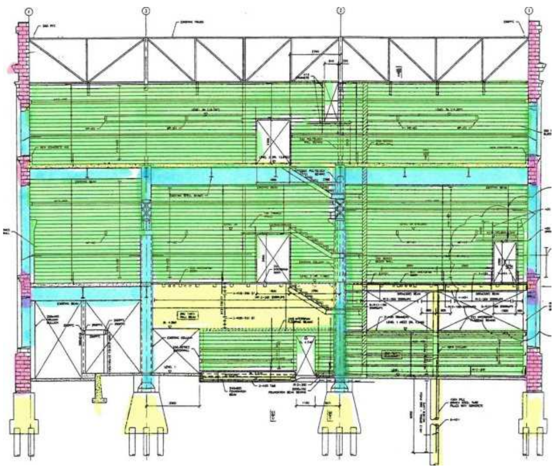
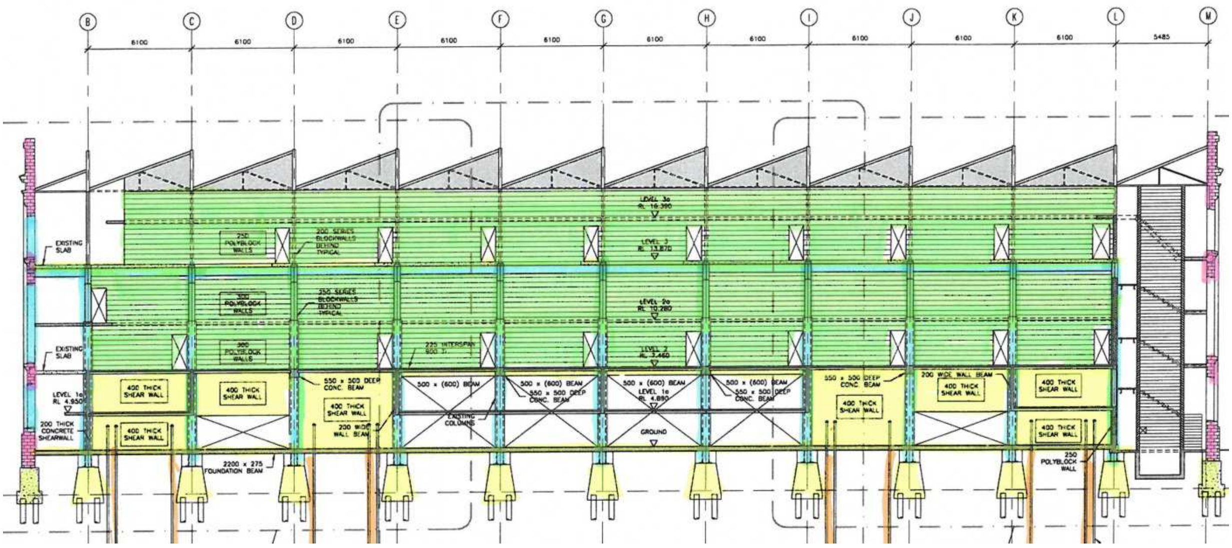
J - K





Grid L

The cross-section below is on grid-line 2 and is the longitudinal seismic resistance system.
As above, it is presented to indicate the considerable scope of the walls that resist seismic loads in the building



5.6 Building Weights and Dimensions

The building has a rectangular plan at 75m (North / South) by 29m (East / West). The footprint is 2175 $m^{2.}$

Refer to the two pages of Dunning Thornton weight calculations following. The weight of the building is a crucial item as, when multiplied by the code seismic coefficient (an acceleration), it gives the lateral load DEMAND on the structure.

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CALCULATIONS

Job Name: SHED 21

Job No.	Calc'd By	Date	Page
3771 3771wt06	las	27-May-01	W1

Building Weight

Dead Load

Roof		1	kPa
Floor	Existing	5.15	kPa
	225 Interspan + topping	**2.5**	kPa
	sdl	0.5	kPa
	Timber floor	0.5	kPa
	Masonry	22	kN/m^3
	Concrete	24	kN/m^3

Seismic Live Load

Domestic	0.6	kPa
Retail	1.6	kPa
Storage/Machine Room	2	kPa

Description	G kPa	Q kPa	Length m	Width m	Area m^2	Gi kN	Qui kN	G + Qui kN
Roof	0.8		61	28	1708	1366.40		1366.40
Framing	0.8		21	13	273	218.40		218.40
	0.8		5	4	20	16.00		16.00
				Σ =	**2001**	**1600.80**	Ÿ=0.46	**1600.80**
3A	0.8	0.6	68	6	408	326.40	119.56	445.96
Floor + sdl	0.8	0.6	62	7	434	347.20	127.18	474.38
	0.8	0.6	6.5	7	45.5	36.40	13.33	49.73
	0.8	0.6	9	5	45	36.00	13.19	49.19
Mach room	2.5	2	7	2.5	17.5	43.75	35.00	78.75
	2.5	2	6	5	30	75.00	60.00	135.00
				Σ =	**932.5**	**864.75**	Ÿ=0.49	**1233.02**
3	5.3	0.6	61	28	1708	9052.40	471.78	9524.18
Ex flr + sdl	5.3	0.6	21	13	273	1446.90	75.41	1522.31
	5.3	0.6	5	4	20	106.00	5.52	111.52
openings	-5.3	-0.6	36	3	108	-572.40	-64.80	-637.20
				Σ =	**2001**	**10032.90**	Ÿ=0.46	**10520.81**
2A	0.8	0.6	62	6.5	403	322.40	119.23	441.63
Floor + sdl	0.8	0.6	61	6	366	292.80	108.29	401.09
	0.8	0.6	6	5	30	24.00	8.88	32.88
	0.8	0.6	7	6	42	33.60	12.43	46.03
				Σ =	**841**	**672.80**	Ÿ=0.49	**921.62**
2	5.3	0.6	74	6.5	481	2549.30	133.09	2682.39
Ex flr + sdl	5.3	0.6	12	7	84	445.20	23.24	468.44
Nw flr + sdl	2.8	0.6	61	21	1281	3586.80	354.46	3941.26
	2.8	0.6	17	6	102	285.60	28.22	313.82
				Σ =	**1948**	**6866.90**	Ÿ=0.46	**7405.92**
1MEZZ	2.8	1.6	6	6	36	100.80	31.13	131.93
Nw flr + sdl	2.8	1.6	24	6	144	403.20	124.53	527.73
	2.8	2	12.5	6.5	81	227.50	87.83	315.33
	2.8	1.6	18	6	108	302.40	93.40	395.80
				Σ =	**369.25**	**1033.90**	Ÿ=0.54	**1370.80**

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CALCULATIONS

Job Name: SHED 21			
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Description	Unit Weight kN/m³	Length m	Height m	Opening m²	Thickness m	Weight kN	Total Weight kN
Roof							
North Wall	22	30	2.25	15	0.457	527.8	
South Wall	22	28	2.25	15	0.457	482.6	
East Wall	22	61	2.25	37.5	0.457	1002.9	
West Wall	22	74	2.25	45	0.457	1221.6	
Parapet	15.4	193	2.25		0.457	3056.2	
Timber Party Walls	3.5	62	2.25		0.15	73.2	
Block Paty Walls	20	242	2.25		0.2	2178.0	
Central Wall	24	62	2.25		0.15	502.2	
Stair Well	22	13	1.4	7.5	0.457	107.6	
Sprayed Concrete	24	5	1.4		0.15	25.2	
Mullions	24	4.8	2.25		0.4	103.7	
Pier	22	44.2	2.25		0.228	498.8	**9779.8**
3							
North Wall	22	30	5.325	42	0.457	1183.9	
South Wall	22	28	5.325	42	0.457	1076.8	
East Wall	22	61	5.325	105	0.457	2210.1	
West Wall	22	74	5.325	126	0.457	2695.0	
Timber Party Walls	3.5	62	5.325	20	0.15	162.8	
East side party walls	20	121	2.25		0.25	1361.3	
	20	121	3.075		0.2	1488.3	
West side party walls	20	100	2.25		0.25	1125.0	
	20	80	3.075		0.25	1230.0	
	24	20	3.075		0.25	369.0	
Central Wall	20	62	3.075		0.2	762.6	
	20	62	2.25	20	0.15	358.5	
Stair Well	22	13	4.45	15	0.457	430.8	
Lift	20	20	5.325		0.2	426.0	
Sprayed Concrete	24	5	4.45		0.15	80.1	
Mullions	24	4.8	5.325		0.4	245.4	
Beams	24	27	0.4		0.2	51.8	
Concrete Band	24	135	0.5		0.2	324.0	
Pier	22	44.2	5.325		0.228	1180.6	**16761.9**
2							
North Wall	22	30	5.725	44	0.457	1284.4	
South Wall	22	28	5.725	44	0.457	1169.3	
East Wall	22	61	5.725	108.5	0.457	2420.2	
West Wall	22	74	5.725	131	0.457	2942.3	
Timber Party Walls	3.5	62	3.075	20	0.15	89.6	
East side party walls	20	121	3.075		0.2	1488.3	
	20	36	2.65		0.2	381.6	
West side party walls	20	80	3.075		0.25	1230.0	
	24	20	3.075		0.25	369.0	
	24	10	2.65		0.2	127.2	
	24	20	2.65		0.25	318.0	
Central Wall	24	108	3.075	20	0.2	1498.1	
	24	36	2.65		0.4	915.8	
Stair Well	22	13	5.725	15	0.457	597.5	
Sprayed Concrete	24	5	5.725		0.15	103.1	
Mullions	24	4.8	5.725		0.4	263.8	
	24	6.4	2.65		0.25	101.8	
Mezzanine Beams	24	50	2.2		0.2	528.0	
	24	60	0.6		0.5	432.0	
Pier	24	44.2	5.725		0.228	1384.7	**17644.6**

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CALCULATIONS

Job Name: SHED 21			
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FLOOR WEIGHT SUMMARY

Level	Floor kN	Walls kN	Total kN
Roof + 1/2 3A	2217.31	9779.78	11997.09
1/2(3A+2A) + 3	11598.13	16761.94	28360.06
1/2(2A+1M) + 2	8552.13	17644.58	26196.71
		Total	**66553.86**

FLOOR MASS SUMMARY

Level	Weight kN	Area m²	Mass kPa
Roof	11997.09	2001.00	6.00
3	28360.06	2001.00	14.17
2	26196.71	1948.00	13.45

The building's seismic mass (66,554 kN) and the individual floor masses are summarised above. The floors, at around 14 kPa, are heavy by most standards due to the heavy concrete walls separating the apartments. However, their acoustic and vibration characteristics are very favourable.

6. Step 2 – Lateral Load Shear Capacity of the Walls

6.1 General Background

The calculations below summarise the seismic mass of the building and define the seismic coefficient. The product of Mass (building mass) and Acceleration (seismic coefficient) give the Lateral Force the building has been designed to resist. Incidentally, this is Newton's Second Law of Physics.

Dunning Thornton calculated the mass of the building at 66,554 kN and the seismic coefficient from the (then) NZS 4203:1992 code is 0.208g.

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CALCULATIONS

Job Name: SHED 21

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3771	las	27-May-01	W4

SEISMIC FORCE DISTRIBUTION

LATERAL FORCE COEFFICIENT:

EQUIVALENT STATIC METHOD - ULTIMATE LIMIT STATE

μ = 3
T= 0
$C_h(T_1,\mu)$ = 0.37 Site Cat = C
Sp = 0.67
R = 1
Z = 1.2
Lu = 0.7

C = 0.208236 V = BASE SHEAR = 13.859 MN

STATIC LATERAL LOAD DISTRIBUTION :

LEVEL	Hi (m)	HT (m)	Wi (KN)	Wi*HT (KNm)	Wi*HT / SUM Wi*HT	Fx (MN)	Vx (MN)	Mx (MNm)
ROOF	4.5	15.95	11997.09	191353.51	0.2922	4.834	4.834	21.753
3	6.15	11.45	28360.06	324722.72	0.4958	6.322	11.156	90.362
2	5.3	5.3	26196.71	138842.57	0.2120	2.703	13.859	163.814
1	0	0	0.00	0.00	0.0000	0.000	13.859	163.814
SUM			6.66E+04	6.55E+05	1.000	13.859		

LEVER ARM = 11.82 m
L.A./HEIGHT = 0.74 (0.70 ?)

The walls were designed to resist a lateral load or shear force of 13,859 kN. This is the seismic mass of 66,553 kN multiplied by the design seismic coefficient of 0.208g.

The ensuing design process which engineers need to perform to satisfy “capacity design” principles generally finishes up with better strengths than the design input initially called for (13,859 kN.)

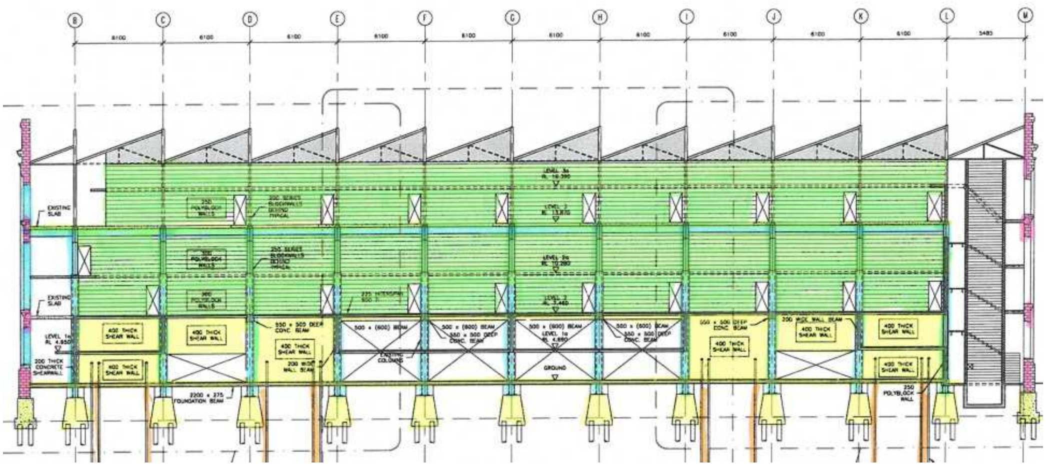
Almost always buildings are stronger in reality than their calculated design load requirement.

However, when assessing buildings for seismic capacity using the NZSEE publication “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes – the Red Book”, one can use an increase in concrete strength of 150% over that designed and 108% of the reinforcing steel strength; these increases are both due to aging.

Both increases have been adopted in the calculations in section 4.2.

The normal capacity reduction factors needed in the new design of all components, including foundations, are not applied to strength **assessment** processes as against strength **design** processes. There are some exceptions to that “rule’ though; one case is when calculating the shear capacity of concrete.

6.2 Shear Capacity of the Wall on Grid-line 2



A study was carried out on the lateral shear capacity, at level 1, of the reinforced concrete shear walls. The following is the calculation of the shear strength of the longitudinal wall on grid 2 shown in elevation above. It covers the four yellowed panels on level 1. These are 400 mm thick concrete walls reinforced with 16 mm diameter high tensile steel reinforcing bars at 400 mm centres on both faces of the wall.

The following calculation is for the 400 thick wall. Full calculations for the other walls are in Section 14 - Appendix D - Calculations for Wall Shear Strength Capacity

Shear Capacity of Reinforced Concrete Walls

Wall grid 2

Number of walls	4	
Wall Thickness b	400	mm
Constant V	0.17	
f'c original design	30	Mpa
Magnification NZSEE Section 7.1.1a)	1.5	
v_c = .17sqrt(f'c)	1.40	Mpa
Bar diameter	16	
Bar Spacing - s	400	
No Bars	2	
Av	402	mm2
fy	450	Mpa
Magnification NZSEE Section 7.1.1a)	1.08	
v_s=Vs/bd=Av*fy*(1/s/b)	1.221	
$v_t = v_c+v_s$	2.62	Mpa
Wall Length	6.1	m
Capacity Reduction Factor - NZSEE Section 7.1.1c)	0.85	
Shear Capacity of Concrete Wall	**5,430**	kN
Total capacity of all 4 walls	**21,720**	kN

One can see that the actual capacity of the four connected walls, at 21,750 kN, is much larger than Dunning Thornton's original design figure of 13,859 kN.

6.3 Summary of the lateral load capacity of all the walls

Reinforced Concrete Walls (transverse) on Grids E,G and I

Number of walls	3	
Wall Grid E	6,083	kN
Wall Grid G	5,489	kN
Wall Grid I	6,379	kN
Total **capacity** reinforced concrete walls	17,951	kN

Reinforced Concrete Walls (Longitudinal) on Grids 2

Number of walls	4	
Total capacity reinforced concrete walls	21,720	kN

The lateral load CAPACITY of the building is therefore governed by the transverse strength of the building at 17,951 kN.

7. Step 3 - Seismic Demand on the Building

Calculations from Dunning Thornton give their seismic design parameters:

LATERAL FORCE COEFFICIENT:

EQUIVALENT STATIC METHOD - ULTIMATE LIMIT STATE

μ =	3		
T=	0		
$C_h(T_1,\mu)$ =	0.37	Site Cat =	C
Sp =	0.67		
R =	1		
Z =	1.2		
Lu =	0.7		

C = 0.208236 V = BASE SHEAR = 13.859 MN

The important values to note are the assumed soil type classification (Type C) and the ductility (μ = 3).

These parameters have been carried into the current AS/NZS 1170.5 code to obtain a current seismic coefficient. The equivalent soil type classification changes from Type C to type D in AS/NZS 1170.5.

The following page is an introduction to Dunning Thornton's design completed in the year 2001. Note bullet point 2; the building was designed to resist 70% of the (then) design code, NZS 4203:1992. Note the value of Lu = 0.7 in the table above. That is the 70% expressed as a reduction factor.

Hence it was never designed to resist 100% of the (then) code.

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CALCULATIONS

JOB NAME	Waterloo on Quay Shed 21		
JOB NO	CALC'D BY	DATE	PAGE
3771	LAS	1-7-2001	1

Introduction

- The following structural calculations are for the seismic strengthening and redevelopment of shed 21 Waterloo Quay, Wellington.The existing building is a three-storey wool store structure, predominantly constructed of brickwork with reinforced concrete ground and second floors on structural steel framing, with light weight roof on steel framing. Timber piles under reinforced concrete pile caps and foundation beams support the structure.

- The building is currently earthquake prone. In addition it is undergoing a change of use from warehouse to residential. Accordingly the building is to be strengthened as near as practical to current codes levels. For this building "as near as is practical" has been assessed as being compliant with %70 of the NZS 4230 – 1992, seismic provisions.

- The analysis and design of new members are in accordance with the latest materials codes, while the existing structure members were analysed using guidelines for strengthening earthquake risk buildings by NZNSEE.

- The design of the building to carry additional floors and walls has entailed the assessment of the capacity and soundness of the existing pile foundations. Included is a report prepared by BCHF regarding the inspection of the piles used to evaluate the adequacy of the existing piles. The calculations show that all the existing piles have sufficient capacity to support the new structure.New driven piles are to be added to assist with the carrying of the additional loads. Test bores have been taken to prove adequate bearing and expected foundation depths.

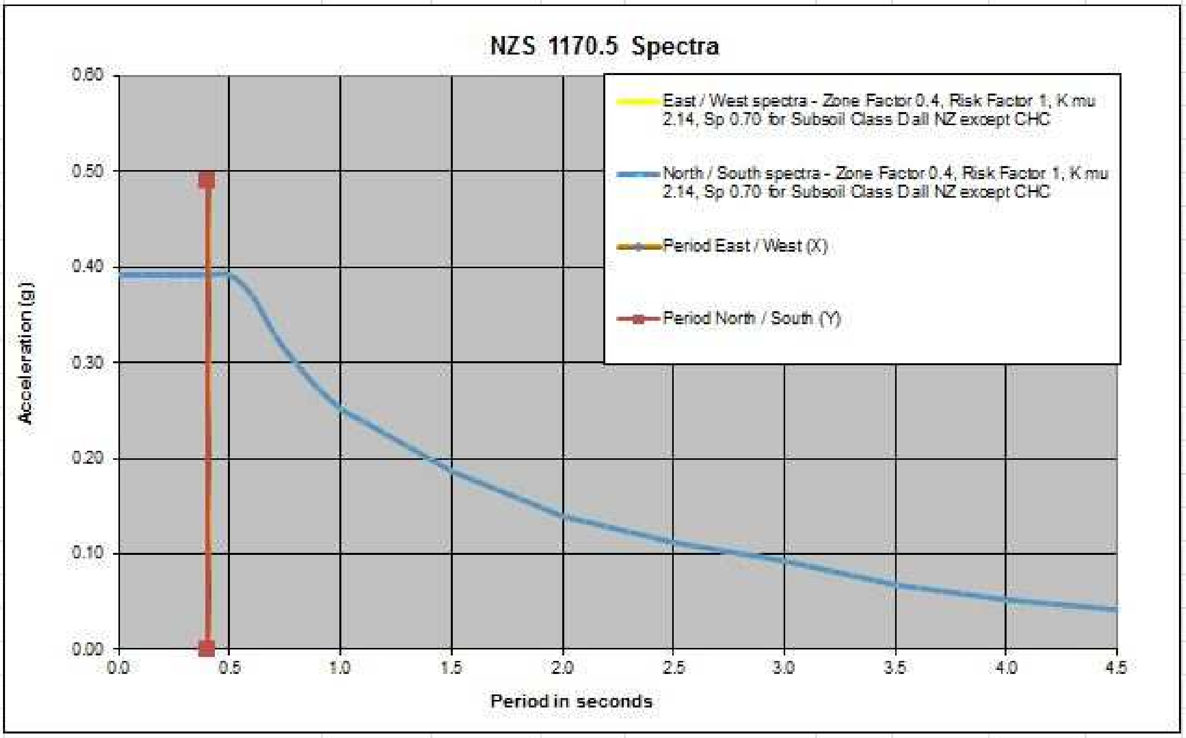
- New shear walls in both directions will act as the prime lateral load resisting elements of the building since the existing external brick walls provide minimal shear capacity for earthquake. New ground anchors will be installed to assist with hold-down for the north south - shear walls.

- The existing building currently has a complete second floor constructed of reinforced concrete. At first floor level there is a concrete gallery floor over approximately 30% of the building area. This floor will be extended to cover the whole area. This floor provides the building platform for the lower level apartments and most importantly provides a floor diaphragm in what is currently a very high space.

Loreta Salazar

The following tables calculate the AS/NZS 1170.5 code seismic coefficient for a 100% NBS case. It is 0.392g. The current code is higher than Dunning Thornton's figure of 0.208g derived from the code they used at the time, NZS 4203:1992, due to:

- Changes in the response spectra
- An increase in Sp (the code's "Structural Performance Factor) from 0.67 to 0.70
- Dunning Thornton designing to 70% of the (then) current code.



Seismic Coefficient Calculator - based on AS/NZS 1170.5

28 Waterloo Quay - Waterloo Apartments - For comparison with Dunning Thornton's calculations 1999 for the seismic upgrade

The Building is in the Wellington CBD zone on Deep or soft soil
It has an importance level 2 for Normal structures and a design life of 50 years

	N/S	E/W	
Periods	0.40	0.40	
Ductility Factors	3.00	3.00	
Near-fault Factors	1.00	1.00	
Structural Performance Factor	0.70	0.70	
Return Period Years - ULS	500	500	
Return Period Years - SLS1	25	25	
Design Acceleration - ULS	0.392	0.392	Cd(T1) - ULS
Design Acceleration - SLS1	0.098	0.098	Cd(T1) - SLS1
Design Acceleration - SLS2	#N/A	#N/A	Cd(T1) - SLS2

Therefore, the base shear **DEMAND** on the building due to the current (AS/NZS 1170.5) code seismic coefficient of 0.392g. when multiplied by the seismic mass of 66,553 kN. gives a **DEMAND** of 26,100 kN.

If one were to simply compare the seismic coefficients used by Dunning Thornton in their design to the NZS 4203:1992 code and the requirements of the current code, one would obtain a value of 54% NBS. Refer to the following table.

However, as above, the actual shear CAPACITY of the wall on Grid 2 is much greater than the design requirement.

Design Comparison	Original Design by Dunning Thornton 1999	New Building Standard 1170.5
Periods	<0.4	<0.4
Ductility Factors	3.00	3.00
Structural Performance Factor	0.67	0.70
Near-fault Factors	N/A	1.00
Reduction factor for existing building Lu	70%	100%
Design Acceleration - ULS	0.210	0.392

Compare original design acceleration with full current code. Retain ductility and period data.	**54%**

8. Step 4 - Determine % NBS

8.1 Lateral load capacity

Refer to section 6 - Step 2 – Lateral Load Shear Capacity of the Walls

Total Capacity of Transverse Reinforced Concrete Walls on Grids E,G and I 17,951 kN.

Total capacity of Longitudinal Reinforced Concrete Walls on Grids 2 21,729 kN.

8.2 Lateral load demand

Refer to section 7 - Step 3 - Seismic Demand on the Building.

The lateral load DEMAND on the building is 26,100 kN

8.3 Resulting %New Building Standard.

The table below, although it repeats information, is intended to clearly show the derivation of the %NBS for both the transverse and longitudinal directions of seismic attack.

Reinforced Concrete Walls on Grids E,G and I

Number of walls	3	
Wall Grid E	6,083	kN
Wall Grid G	5,489	kN
Wall Grid I	6,379	kN
Total **capacity** reinforced concrete walls	17,951	kN
Lateral Load **Demand** at mu = 3	26,100	
Lateral Load **Capacity** divided by Lateral Load **Demand**	**69%NBS**	

Reinforced Concrete Walls on Grids 2

Number of walls	4	
Total capacity reinforced concrete walls	21,720	kN
Lateral Load **Demand** at mu = 3	26,100	
Lateral Load **Capacity** divided by Lateral Load **Demand**	**83%NBS**	

Hence the lower value, at 69% NBS gives the rating on the actual structure of the building.

9. Effect of the Geotechnical "Spreading" of the site on the %NBS

9.1 Current position

Geotechnical studies on the stability of the site have been completed and are presented in section 16 - Appendix F – Geotechnical Seismic Stability of the Site. This indicates that the site itself becomes a partial "weak link" beginning approximately 50% NBS, due to the reclamation fill tending to "stretch out" across the site between Waterloo quay and the sea wall under severe seismic shaking.

This starting point of 50% NBS is based on Tonkin and Taylor's expected lateral movement at the south eastern side of the building of between 50 and 300mm The building should be able to tolerate that movement without collapse.

At 75% NBS, Tonkin and Taylor have given an expected outcome of a lateral movement at the south eastern side of the building of "hundreds of millimetres".

At some point the building will not be able to tolerate a large extension or "stretch" across its floor plate and the outer columns. Dunning Thornton's design has added reinforced concrete columns cast against the back of each brick pier. The columns are 700mm wide by 250mm deep (deep means protruding into the building)

These concrete columns will provide a large measure of residual strength to the eastern brick piers if they have their bases pulled towards the sea wall by the ground movement.

Based on the range of movements estimated by Tonkin and Taylor in Table 1 of their report we, as structural engineers consider the building should be able to tolerate ground movements that could be expected to result from between 50% and 60% NBS shaking. An average may be 55%; let us settle on that figure for this report. As stated previously, the science of geotechnical analyses such as presented in section 16 is not exact and some finite judgement call is needed to advance this global assessment.

Consequently, unless the ground is remediated, the **global percentage NBS of the building needs to be downgraded to 55%NBS** to recognise the potential detrimental effect of the lateral spreading across the site.

9.2 Potential Ground Remediation

Tonkin and Taylor's stage 2 work (presented in section 16 - Appendix F – Geotechnical Seismic Stability of the Site) has been completed.

If the Body Corporate chooses to enhance the capacity of the soil material on the seaward side of the building to improve its %NBS performance, then geotechnical studies will need to be carried out to design a sensible and cost effective solution to improve the lateral load performance of that part of the site. Stuart Palmer of Tonkin and Taylor has verbally mentioned "soil mixing" as a potential remedy.

The actual building lies at 69% NBS; therefore the ground improvement would need to equal or better that value to increase the global %NBS of the apartment block.

10. The New Zealand Society for Earthquake Engineering Grading Scheme

10.1 Extract from NZSEE outlining the scheme

In addition to the legislative requirements, the NZSEE is keen to introduce into the property market a system for grading buildings according to their assessed structural performance. The aim is to raise awareness in the industry and allow market forces to work in reducing earthquake risk. In time, owners of lowest grades of buildings would find themselves under pressure to improve them or face loss of revenue.

Table 2.1 indicates the grading scheme proposed. This is linked to the %NBS value. Determining the earthquake risk grade of a building would be a simple matter of determining into which grade band the calculated %NBS of the building falls.

Note that the grade is not required by the Act, but is seen by NZSEE as a highly desirable mechanism to bring about improvement of structural performance.

Table 2.1 includes an indication of the relative risk for buildings designed at different times. The relative risk represented by the progressively decreasing %NBS shows the importance of dealing with those buildings with less than or equal to 33%NBS – they have 20 or more times the risk of their strength being exceeded due to earthquake actions.

Table 2.1: Grading system for earthquake risk

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					
67–80	B	2–5 times					
33–67	C	5–10 times					
20–33	D	10–25 times					
<20	E	> 25 times					

Note changes to the relative risk values have been made to line up with the values in Table C4.4.

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 requriements 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.

The NZSEE Study Group sought to summarise its views on how buildings of various risk levels should be regarded. The result is shown in Table 2.2. Buildings that do not comply with the minimum requirements of the proposed changes to the Act (i.e. ≤33*%NBS*) are regarded as High Risk Buildings. Those with > 67*%NBS* are regarded as being Low Risk. This leaves a group in between that meet the requirements of the Act but cannot be regarded as Low Risk. These have been termed Moderate Risk.

These definitions differ from the requirements of the Act. The Act requires that buildings be improved to at least 34*%NBS*. Table 2.2 indicates the difference.

Table 2.2 NZSEE Risk Classifications and Improvement Recommendations

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance: Legal Requirement	Improvement of Structural Performance: NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	Unacceptable	Unacceptable

10.2 Resulting Building Seismic Assessment

The building itself, ignoring any potential geotechnical issues, is assessed at 69% NBS. Thus the building has a “B Grade” rating from NZSEE table 2.1 and is a Low Risk in terms of table 2.2.

The stability of the site has been assessed by Geotechnical Engineers – Tonkin and Taylor and the interpolation of their results, married with the likely performance of the building when exposed to the movement in the ground tending to :stretch” it across its width, results in a diminution of the seismic rating to approximately 55% NBS.

Needless to say, the building came through the Sunday 21st July Cook Strait earthquake and its aftershocks with no structural damage. That earthquake was equivalent to approximately 60% NBS.

Report prepared by:

Peter Johnstone

Peter Johnstone Consulting Engineer

BE (Hons) PhD (Civil) FIPENZ MRSNZ MNZIOB

Revision 04 – 9 March 2014

11. Appendix A - Geotechnical Data Circa 2000

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ISO 9001

Ref: 3771L06

8 August 2000

Newcrest Holdings Ltd
PO Box 2609
WELLINGTON

FAXED

Attention: Steve Rowe

Dear Steve

Shed 21 Test Bores

Attached are the bore logs for the two test bore holes carried out inside Shed 21 last week.

At the Northern End (BH1) adequate pile bearing should be obtained at between 7 to 8.5m.

At the Southern End (BH2) adequate pile bearing should be obtained at around 9m.

All piles will be required to achieve an adequate set to "prove" a safe working load.

Yours faithfully

Adam Thornton
DIRECTOR

A2

WEBSTER DRILLING & EXPLORATION LTD

DRILLERS BORE LOG

0597

CLIENT:	Dunning Thornton Consultants Ltd	BORE No.	Two
CONTRACT:		DATE STARTED:	3 - 8 - 00
LOCATION:	Shed 21	DATE FINISHED:	5 - 8 - 00
RIG USED:	HE	OBJECTIVE:	Geotech Investigation
DRILLING METHOD:	Rotary mud	DRILLER:	G. Kingan

GEOLOGICAL LOG

FROM	TO	DESCRIPTION	Bed Thickness
Floor	200mm	Concrete	
.2m	3·7m	Light brown silty clay and gravels	
3·7m	4·3m	Grey silty clay and gravels	
4·3m	4·8m	Brown silty gravels	
4·8m	5·7m	Grey brown silty clay and gravels	
5·7m	8·1m	Black grey silty clay with some black sands	
		and shells	
8·1m	8·4m	Green grey silty sandy gravels	
8·4m	8·9m	Green grey silty sand	
8·9m	10m	Green grey silty sandy gravels	

RECEIVED
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07 AUG 2000
CAD | AWT | DELOG
JOB NO. 3771 | INITIALS Lm

CASING

FROM	TO	DIA
0m	2·5m	4"

MUDS USED

TYPE	QUANTITY

COMMENTS

aphic 38851

WEBSTER DRILLING & EXPLORATION LTD

A3

DRILLERS BORE LOG

0596

CLIENT:	Dunning Thornton Consultants Ltd	BORE No.	One
CONTRACT:		DATE STARTED:	2 - 3 - 00
LOCATION:	Shed 21	DATE FINISHED:	3 - 3 - 00
RIG USED:	H.E.	OBJECTIVE:	Geo-tech Investigatio
DRILLING METHOD:	Retary mud	DRILLER:	G. Kingan

GEOLOGICAL LOG

FROM	TO	DESCRIPTION	Bed Thickness
Floor	200mm	Concrete	
2m	5.7m	Brown silty clay and gravels, some brick fragments	
		to 2.5m	
5.7m	6.4m	Blue grey sandy silt with sea shells and gravels	
6.4m	7.3m	Grey silty sand and gravels	
7.3m	8.4m	Light brown sandy silt and gravels	
8.4m	8.7m	Grey sandy gravels	
8.7m	9.5m	Light brown sandy silt and gravels	
9.5m	10m	Brown and grey mottled sandy silt and gravels	

CASING

FROM	TO	DIA
0m	6m	4"

MUDS USED

TYPE	QUANTITY

COMMENTS

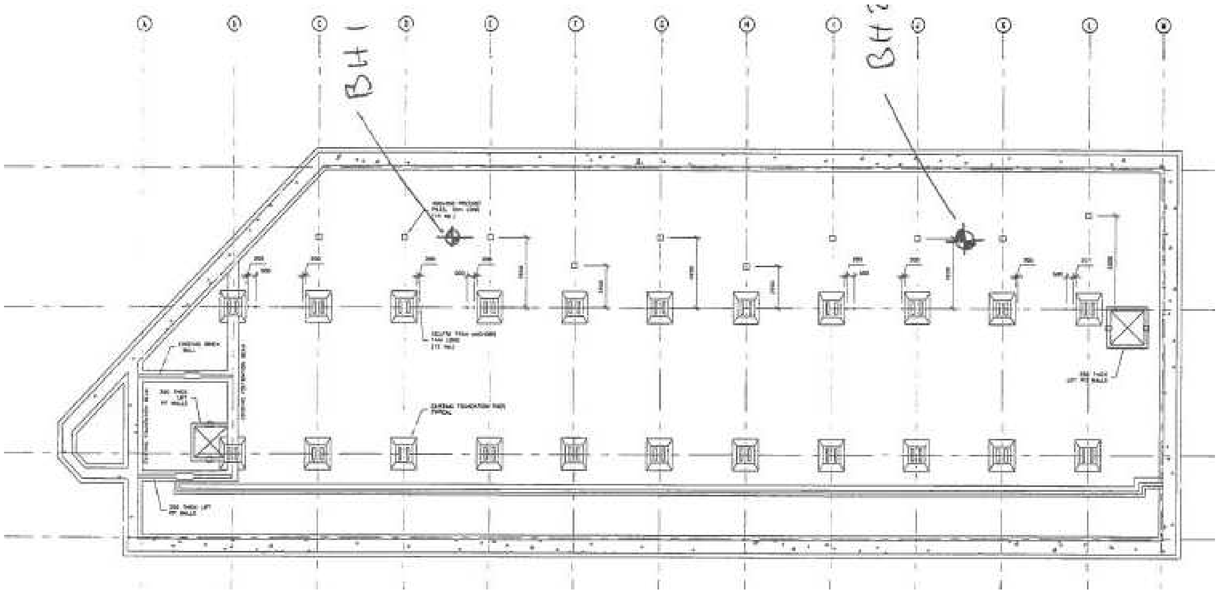
iraphic 36861

A4

Dunning Thornton Consultants Ltd   Shed 21   Geo tech Investigation

SPT blow counts   Boreholes one and two   03-08-00 — 04-08-00

		(N)			(N)
BH#1 1m	3//1/1/1/2	(5)	BH#2 1m	4//2/2/2/2	(8)
2m	2//1/2/2/1	(6)	2m	2//2/1/1/1	(5)
3m	4//2/2/1/2	(7)	3m	3//2/2/2/4	(10)
4m	5//2/2/2/3	(9)	4m	3//3/1/2/1	(7)
5m	5//3/2/3/2	(10)	5m	3//2/2/2/2	(8)
6m	8//5/5/7/7	(24)	6m	2//1/2/3/4	(10)
6.5m	9//3/2/5/3	(18)	6.5m	10//3/5/7/8	(23)
7m	6//1/8/7/8	(24)	7m	9//3/3/5/5	(16)
7.5m	12//6/8/8/8	(30)	7.5m	5//1/2/3/2	(8)
8m	12//6/8/10/11	(35)	8m	8//8/8/4/4	(24)
8.5m	17//9/9/12/11	(41)	8.5m	3//3/2/3/3	(11)
9m	14//11/12/13/15	(51)	9m	6//7/10/11/16	(44)
9.5m	17//10/10/16/14	for 55mm $(50)^+$	9.5m	12//10/20/15/5	for 50mm $(50)^+$
10m	17//13/13/11/14	for 70mm $(51)^+$	10m	16//11/12/17/10	for 25mm $(50)^+$



12. Appendix B – BECA Report on Existing Piles

AG

5

BECA

Our Ref: 3400025
IWL98322.DOC

16 July 1998

Lambton Harbour Management Ltd
PO Box 395
WELLINGTON

Attention: Mr Bruce Green

Dear Sir

SHED 21 PILES INSPECTION

1 Introduction

Lambton Harbour Management Ltd asked Beca Carter Hollings & Ferner (BCHF) to arrange a test pit to uncover the top of two piles, to allow inspections by potential developers and for BCHF to inspect the piles and report on their condition. BCHF was also required to indicate the likely loading that the piles can be subjected to.

The bearing capacity of the piles has been previously considered by Beca Carter Hollings & Ferner Ltd (BCHF) and our conclusions are given in a letter-report dated 11 June 1993 addressed to Mr Graeme Patterson. The conclusions of the 1993 BCHF report were based on our knowledge of ground conditions, historical construction details, past performance of the piles and inspection of the physical condition of one pile in a test pit (the location of the 1993 test pit is shown on Figure 1).

The current test pit investigations have been undertaken to give potential developers an opportunity to inspect the piles (and their pile cap) and to obtain more information on the physical condition of the piles. On 30 June 1998 a test pit was excavated on the seaward (east) side of Shed 21, at 5 m distance from the north end of the building (refer Figure 1) to expose the top of two piles and inspect their condition. The excavation work was undertaken by N Forsyth Ltd. The conditions encountered were observed and recorded by a BCHF geotechnical engineer. The services of Mr G Painter, Tranz Rail New Zealand bridge inspector were also called upon to inspect the piles. (As Mr Painter routinely inspects hardwood timber members his experience and expertise was considered valuable in assessing the condition of the timber piles).

Several potential developers invited by Lambton Harbour Management were also given an opportunity to inspect the piles.

Beca Carter Hollings & Ferner Ltd
77 Thorndon Quay, Wellington 1, New Zealand. P O Box 3942, Wellington, New Zealand.
Telephone +64-4-473 7551 Fax +64-4-473 7911

A7

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Our Ref: 3400025
1WL98322.DOC

Page 2
16 July 1998

Our observations and conclusions from the inspection are reported below. Figure 2 attached shows in cross section the profile observed in the test pit.

2 Observations

2.1 Ground Conditions

Shed 21 is located over reclamation fill placed during the period 1901 to 1903. Reclamation fill was encountered in the test pit comprising loose sandy gravel. Similar ground conditions were encountered by the 1993 test pit. Our expectation is that the fill extends to 6 m - 8 m depth and overlies beach sand and/or alluvium. Historical drawings indicate the piles to be 7.5 m long and the pile caps to be approximately 3 m below ground surface level. As expected, groundwater level followed the tide level. The groundwater level encountered in the test pit was drawn down by a pump to allow inspection of the pile.

2.2 Ground Beam and Pile Cap

The dimensions of the concrete ground beam and pile cap recorded in the test pit are shown on Figures 2 and 3. There was no evidence of cracking or degradation of the concrete.

2.3 Pile Condition

The exposed sections of two piles were visually inspected, probed and each had a hole drilled through it by a Tranz Rail NZ bridge inspector (refer cross section 1 on Figure 2). From his inspection, the bridge inspector reports the piles comprised sound ironbark hardwood of 380 mm in diameter. The boring indicated no trace of discolouring or decay. Therefore, the two piles are considered to be in sound condition.

The pile inspected by BCHF in 1993 had a trace of discolouring of the timber fibre near the centre of the pile. While this discolouring was an indication of decay, the possible decay was considered to be minor and not of concern with respect to the strength and durability of the pile.

Most decay of timber piles occurs as a result of alternate wetting and drying. This is likely to be a deliberate feature of the original design that the level of the pile cap is such that the piles remain permanently below water and therefore are never exposed to the air to enable them to dry out to protect the piles from decay. However in the absence of as built data it is not possible to state that all pile caps extended to below LWS. The soundness of the three piles inspected (one pile in 1993 and two piles in 1998), suggests it is likely to assume that piles supporting Shed 21 will be in a sound condition where they have pile caps below LWS. While we have not inspected the building in detail, the

BECA

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Our Ref: 3400025
IWL98322.DOC

Page 3
16 July 1998

absence of cracking or other evidence of significant differential settlement of the existing building suggests that no settlement has occurred from failure of piles by decay.

3 Bearing Capacity

It was assessed in our 1993 report that the existing building applies dead plus live loads of between 240 and 300 kN/pile. We are not aware of any differential settlement problems associated with the existing building. The absence of any evidence of differential settlements suggests that piles have performed adequately under this loading.

Our assessment, based on this past performance and assuming that the piles are driven into compact alluvium with properties similar to those encountered by historical boreholes drilled for the Gateway project and Lambton Tower Project and located 250-260m away, suggests that these piles could support a 20% increase in load (ie up to 360 kN/pile dead load plus live load) without inducing differential or total settlements greater than 10 mm.

1080 kN/PILE ULT

BCHF would expect any developer to confirm the conditions by appropriate site investigations and/or proof loading before adopting bearing capacity value. Based on results of site investigations and/or proof loading it may be possible to justify load capacities higher than 360 kN/pile. It is possible that proof loading could demonstrate a safe working load of up to 450 kN/pile.

1350 kN/PILE ULT

4 Conclusions

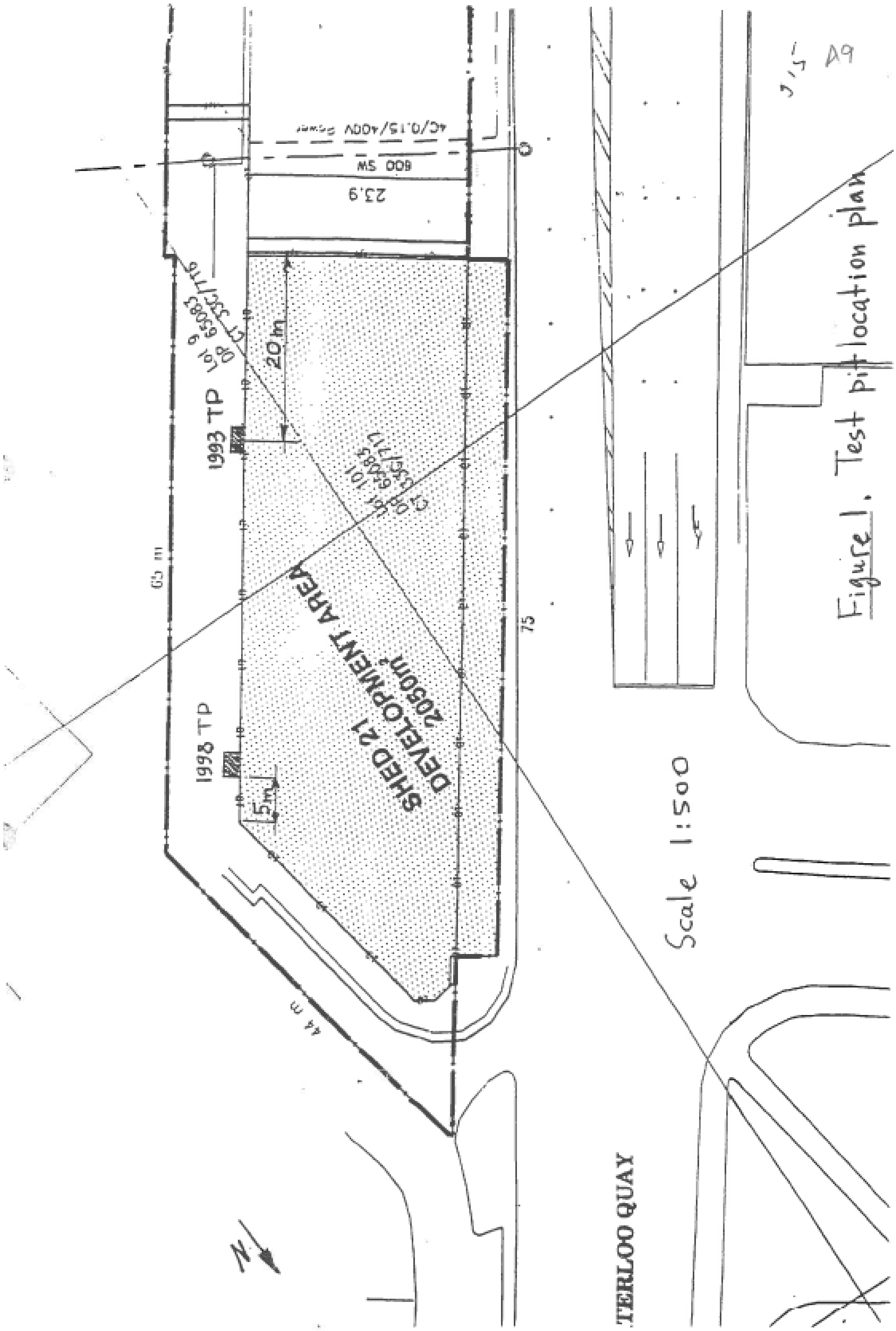
The obtained 1998 investigation data is consistent with the 1993 investigation data.

On the basis of our investigations we conclude that it can be reasonably assumed that the piles supporting Shed 21 are in sound condition.

Yours faithfully
BECA CARTER HOLLINGS & FERNER LTD

ALEXEI MURASHEV

akm:murashak


A9
4C/0.15/400V Power
600 SW
23.9
Lot 9 DP 65083 CT 35C/716
1993 TP
20 m
Lot 10 DP 65085 CT 35C/717
SHED 21 DEVELOPMENT AREA
2050m²
1998 TP
5m
65 m
75
44 m
Scale 1:500
Figure 1. Test pit location plan
TERLOO QUAY

A10

Beca Carter Hollings & Ferner

BECA

CALCULATION SHEET

JOB NAME: Shed 21

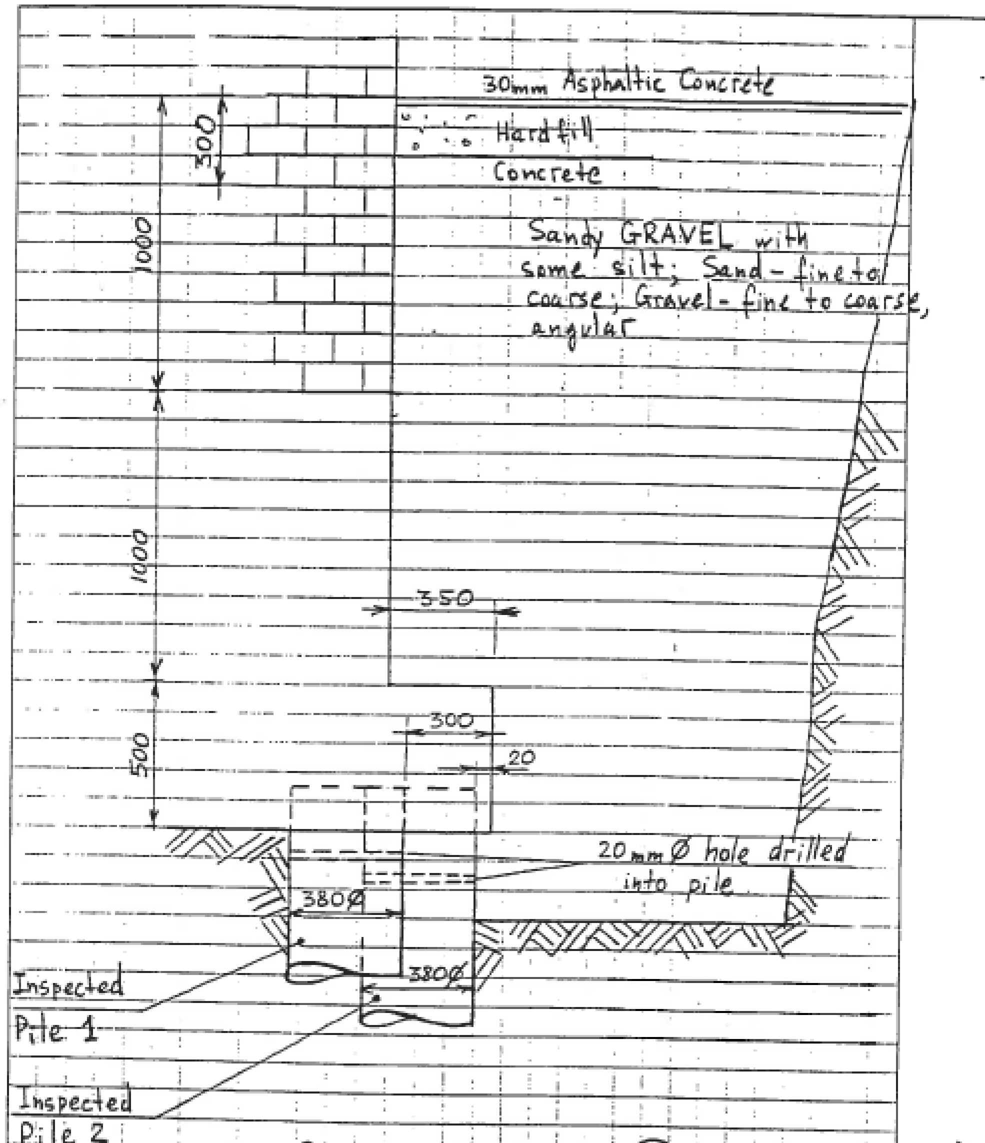
SUBJECT: Pile Foundation – Cross Section 1

BY: AKM

JOB NO: 3400025

PAGE NO: 1 OF 2

DATE: 8/7/98



Cross Section 1 (Scale 1:20)

Figure 2

Beca Carter Hollings & Ferner

BECA

A19

CALCULATION SHEE

JOB No: Shed 21

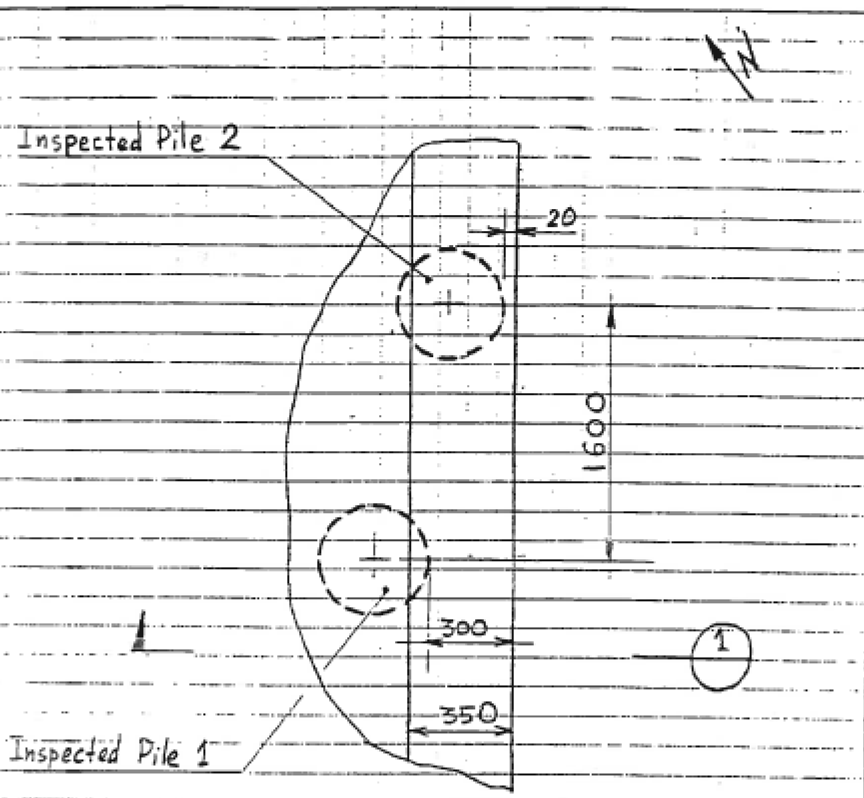
SUBJECT: Pile Foundation - Plan

BY: AKM

JOB NO: 3400025

PAGE NO: 2 OF 2

DATE: 8/7/98



Plan (Scale 1:20)

Figure 3

13. Appendix C – BECA Timber Pile Inspection

A11

BECA

COPY

2704846

11 June 1993

Lambton Harbour Management Ltd
PO Box 395
WELLINGTON

14/6.

Attention: Mr Graeme Patterson

Dear Sir

SHED 21: TIMBER PILE INSPECTION

1 BACKGROUND

In our letter to you dated 24 February 1993, we reported we had considered the bearing capacity of the existing piles beneath Shed 21, with respect to the increased loading to be imposed by the proposed refurbishment. On the basis of historical construction details of the piles, their past performance and our knowledge of ground conditions in this area, we concluded the existing piles can be safely assumed to have adequate bearing capacity to support the additional loads imposed by the proposed refurbishment. It was assessed the existing building applies dead plus live loads of between 24 and 30 tonne/pile, while the refurbished building will apply dead plus live loads of between 24 and 36 tonne/pile. Load testing of the existing piles was not considered necessary. However, it was considered necessary to check the physical condition of the piles to ensure they have not greatly deteriorated over their 80 year life. We proposed that a test pit be excavated to inspect the top of one of the piles.

In your letter dated 23 April 1993, you instructed us to proceed with the proposed test pit. This letter reports on the investigation work which found the top of the pile inspected to be in sound condition.

2 INTRODUCTION

On 27 May 1993 a test pit was excavated on the seaward side of Shed 21 to expose the top of a pile and inspect its condition. The excavation work was undertaken by N Forsyth Ltd and the conditions encountered were observed and recorded by a Beca Carter Hollings & Ferner (BCHF) geotechnical engineer. The services of a New Zealand Rail (NZR) bridge inspector were also called upon to inspect the pile. As these inspectors routinely inspect hardwood timber members their experience and expertise was considered valuable in assessing the condition of the timber pile.

.../

THE GOVERNOR-GENERAL'S SUPREME AWARD FOR EXCELLENCE IN EXPORT ACHIEVEMENT

AN AIR NEW ZEALAND EXPORT EXCELLENCE AWARD

Beca Carter Hollings & Ferner Ltd □ Consulting Engineers

77-79 Thorndon Quay, Wellington 1, New Zealand. P.O. Box 3942, Wellington 1, New Zealand.
Telephone: 0-4-473 7551. Fax: 0-4-473 7911.

A12

BECA

Page 2

11 June 1993

Our observations, and conclusions from the inspection are reported below. Figure 1 attached shows in cross-section the profile observed in the test pit. The test pit was located on the seaward (east) side of shed 21 and 20m from the south end of the building.

3 GROUND AND WATER CONDITIONS

Shed 21 is located over reclamation fill placed during the period 1901 to 1903. Reclamation fill was encountered in the test pit comprising loose sandy gravel. The groundwater level encountered in the test pit was drawn down by a pump to allow inspection of the pile. As expected, groundwater level followed the tide level.

4 GROUND BEAM AND PILE CAP

The dimensions of the concrete ground beam and pile cap recorded in the test pit approximately agree with the original construction drawings. There was no evidence of cracking or degradation of the concrete.

The underside of the pile cap is recorded on the original construction drawings to be positioned at a Reduced Level RL -0.6m (new city datum). Although no survey was undertaken to determine absolute levels of the base of the pile cap, on the basis of estimated ground levels and the recorded depth of 3.0 m from ground level to the base of the pile cap, we conclude that the pile cap has been constructed approximately as detailed on the original construction drawings. Mean low water spring tides are at RL -0.4m (compare base of pile cap at RL -0.6m), indicating that the top of the piles remain under water at all times.

5 PILE CONDITION

The exposed section of pile was visually inspected, probed and had a hole drilled through it by a NZR bridge inspector. From his inspection, the bridge inspector reports the pile comprised sound ironbark hardwood of 380mm in diameter. The boring indicated a trace of discolouring of the timber fibre near the centre of the pile. This discolouring is an indication of decay, but the possible decay was considered to be minor and not of concern with respect to the strength and durability of the pile. 6m long

Most decay of timber piles occurs as a result of alternate wetting and drying. As indicated in 4.0 above, the level of the pile cap is such that the piles remain permanently below water and are never exposed to the air to enable them to dry out. This is a deliberate feature of the original design to protect the piles from decay. Based on this fact and the soundness of the one pile inspected, it is considered reasonable to assume that all piles supporting Shed 21 are in a sound condition. The absence of cracking or other evidence of differential settlement of the existing building supports this conclusion.

BECA

Page 3

11 June 1993

6 STORMWATER PIPE

A 100mm diameter ceramic stormwater pipe was encountered and damaged during the excavation of the test pit (Refer Figure 1). It is assumed the pipe collects water from the building's downpipes. Temporary reinstatement of the pipe was provided during backfilling of the pit, however it is noted the pipe was completely blocked with silt preventing it from acting as a drainage pipe. The pipe will require replacement during any permanent refurbishment work.

7 CONCLUSION

As a consequence of our investigations, we conclude it can be reasonably assumed the piles supporting Shed 21 are in sound condition and have adequate capacity to reliably support the loading to be imposed by the proposed refurbishment.

Yours faithfully
BECA CARTER HOLLINGS & FERNER LTD

M. Chalmers

M CHALMERS

sp:plj
LETPLJ.LBM

14. Appendix D - Calculations for Wall Shear Strength Capacity

Wall grid E

Wall Thickness b	250	mm
Constant V	0.17	
f'c original design	30	Mpa
Magnification NZSEE Section 7.1.1a)	1.5	
v_c = .17sqrt(f'c)	1.40	Mpa
Bar diameter	12	
Bar Spacing - s	300	
No Bars	2	
Av	226	mm2
fy	450	
Magnification NZSEE Section 7.1.1a)	1.08	
v_s=Vs/bd=Av*fy*(1/s/b)	1.466	
v_t = v_c+v_s	2.86	Mpa
Wall Length	10	m
Capacity Reduction Factor - NZSEE Section 7.1.1c)	0.85	
Shear Capacity	6,083	kN

Wall grid G

Wall Thickness b	200	mm
Constant V	0.17	
f'c original design	30	Mpa
Magnification NZSEE Section 7.1.1a)	1.5	
v_c = .17sqrt(f'c)	1.40	Mpa
Bar diameter	12	
Bar Spacing - s	300	
No Bars	2	
Av	226	mm2
fy	450	
Magnification NZSEE Section 7.1.1a)	1.08	
v_s=Vs/bd=Av*fy*(1/s/b)	1.832	
v_t = v_c+v_s	3.23	Mpa
Wall Length	10	m
Capacity Reduction Factor - NZSEE Section 7.1.1c)	0.85	
Shear Capacity	5,489	kN

Wall grid I

Wall Thickness b	275	mm
Constant V	0.17	
f'c original design	30	Mpa
Magnification NZSEE Section 7.1.1a)	1.5	
v_c = .17sqrt(f'c)	1.40	Mpa
Bar diameter	12	
Bar Spacing - s	300	
No Bars	2	
Av	226	mm2
fy	450	
Magnification NZSEE Section 7.1.1a)	1.08	
v_s=Vs/bd=Av*fy*(1/s/b)	1.332	
$v_t = v_c+v_s$	2.73	Mpa
Wall Length	10	m
Capacity Reduction Factor - NZSEE Section 7.1.1c)	0.85	
Shear Capacity	6,379	kN

15. Appendix E – Synopsis of the Building Code

All building work in New Zealand must comply with the Building Code, which is the first schedule to the Building Regulations 1992.

The Building Code does not prescribe how work should be done but states, in general terms, how the completed building must perform in its intended use. The Building Code contains functional requirements and performance criteria that cover matters such as protection from fire, structural strength, moisture control and durability.

Building plans and specifications are assessed by building consent authorities (usually the local territorial authority) to ensure that the completed building work comply with the Building Code. When the building consent authority is satisfied, it will issue a building consent for the work to proceed.

The Building Code consists of two preliminary clauses and 37 technical clauses. Each technical clause, except for the 'C' Clauses for protection from fire, contains:

Objectives - The social objective that completed building work must achieve

Functional requirements - What the completed building work must do to satisfy the social objective

Performance criteria - Qualitative or quantitative criteria with which buildings must comply in their intended use.

Clause C1 contains the Objectives for the Protection from Fire Clauses C2 to C6. Clauses C2 to C6 contain only Functional Requirements and Performances.

The Building Code was reviewed to align it with the 2004 Act

16. Appendix F – Geotechnical Seismic Stability of the Site

Tonkin & Taylor

T&T Ref: 85642.001
24 February 2014
Revision 1

Waterloo on Quay Body Corporate 309984
C/- Waterfront Studio 1.02, 28 Waterloo Quay
PO Box 2282
Wellington

Attention: Paul Harper

Dear Paul

Shed 21, 28 Waterloo Quay, Pipitea, Wellington

Geotechnical Seismic Stability

This letter and other Tonkin & Taylor Ltd letters and reports for this project discuss predicted damage to the ground at the Shed 21 site as a result of possible earthquake shaking. Predicted damage to the building is not discussed. Structural engineer Peter Johnstone is separately reporting on expected performance of the building.

Performance at 34% NBS

Previously T&T has provided two letter reports for this project. Namely:

- Letter dated 22 May 2013
  "Geotechnical Seismic Desktop Assessment Report"
- Letter dated 10 February 2014
  Response to Wellington City Council's request for further information

These letters reported on our assessment of the likely performance of the site in the event of an earthquake which is 1/3 as intense and of similar duration to the earthquake which is currently allowed for in the design of new buildings i.e. 34% NBS (NBS: New building standard).

These letters conclude that 34% NBS earthquake shaking is not likely to result in ground damage at the Shed 21 site. The basis of this conclusion is that the site is likely to have felt more intense earthquake shaking in the past without ground damage. The available information indicates that the intensity of shaking felt as a result of earthquakes in 1942 (Masterton Earthquakes) and 2013 (Cook Strait and Lake Grassmere Earthquakes) had a similar or higher intensity than a 34% NBS event, and these earthquakes did not result in any known ground damage at the site of Shed 21.

A level of shaking equivalent to 34% NBS or greater could be expected to be felt at the site once every 50 years on average. Or a 20% chance of it occurring in the next 10 years.

Tonkin & Taylor Ltd - Environmental and Engineering Consultants, ASB Tower, 2 Hunter Street, Wellington 6011, New Zealand
PO Box 2083, Wellington, Ph: 64-4-381 8560, Fax: 64-4-381 2908, Email: well@tonkin.co.nz, Website: www.tonkin.co.nz

Performance at greater than 34% NBS

The purpose of this letter is to discuss the predicted performance of the site in the event of earthquakes causing more intense shaking at the site than 34% NBS.

Figure 2 attached from our 23 May 2013 letter summarises the inferred ground conditions at the site. The site is on reclaimed land with an approx. 7m high concrete seawall forming the edge of the reclamation less than 10m south east of the Shed 21 building. A wharf structure abuts the seawall (Refer Figure 1 and 2 attached).

Table 1 summarises predicted ground movements at the south eastern side of Shed 21 as a consequence of various levels of earthquake shaking. Differential lateral ground movements beneath the south eastern side of the building relative to the remainder of the building could be equal to the movements recorded in Table 1. The ability of the building piles to resist these lateral movements should be assessed by the structural engineer in consultation with the geotechnical engineer.

Table 1. Predicted Lateral Ground Movements

Earthquake Shaking			Predicted Ground Movement at SE Side of Shed 21		
% NBS	Return Period	Probability of Occurring in Next 10 Years	Estimated favourable outcome. Possible but not likely	Expected outcome	Estimated unfavourable outcome. Possible but not likely.
34%	50 Years	20%	<10mm	<10mm	50mm
50%	100 Years	10%	<10mm	50mm – 300mm	100's mm
75%	250 Years	4%	50mm	100's mm	> 1m
100%	500 Years	2%	100's mm	>1m	> 1m

The conclusions presented in Table 1 are based on the following:

- **Liquefaction**
  The potential for a soil to liquefy is dependent on the intensity of shaking felt and the nature of the soil. Based on the information currently available liquefaction is possible but not expected at 50% NBS or less. At 75% NBS or more, liquefaction could be expected. We cannot discount the possibility of the fill not liquefying at these intense levels of shaking (>75% NBS). If the reclamation fill were to liquefy as a result of earthquake shaking, it is likely that the seawall would slide or rotate resulting in large ground movements beneath Shed 21. In the possible but unlikely event that the seawall does not slide or rotate as a result of liquefaction, cyclic displacements of the order of 300mm of the liquefied fill could be expected during the shaking.
- **Seawall Displacements**
  As discussed above, if liquefaction occurs, sliding or rotation of the seawall would be expected. If liquefaction does not occur, with intense shaking (not likely but possible), the earthquake shaking could still be expected to cause rotation or sliding of the seawall, resulting in large ground movement beneath Shed 21.

  Assuming the wharf provides minimal restraint to the top of the seawall, rotation of the seawall is expected at 50% NBS and greater even if liquefaction does not occur.

Applicability

Recommendations and opinions in this report are based on data from two boreholes recorded by the driller. The nature and continuity of subsoil away from the boreholes are inferred and it must be appreciated that actual conditions could vary from the assumed model.

This report has been prepared for the benefit of Waterloo on Quay Body Corporate 309984 with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

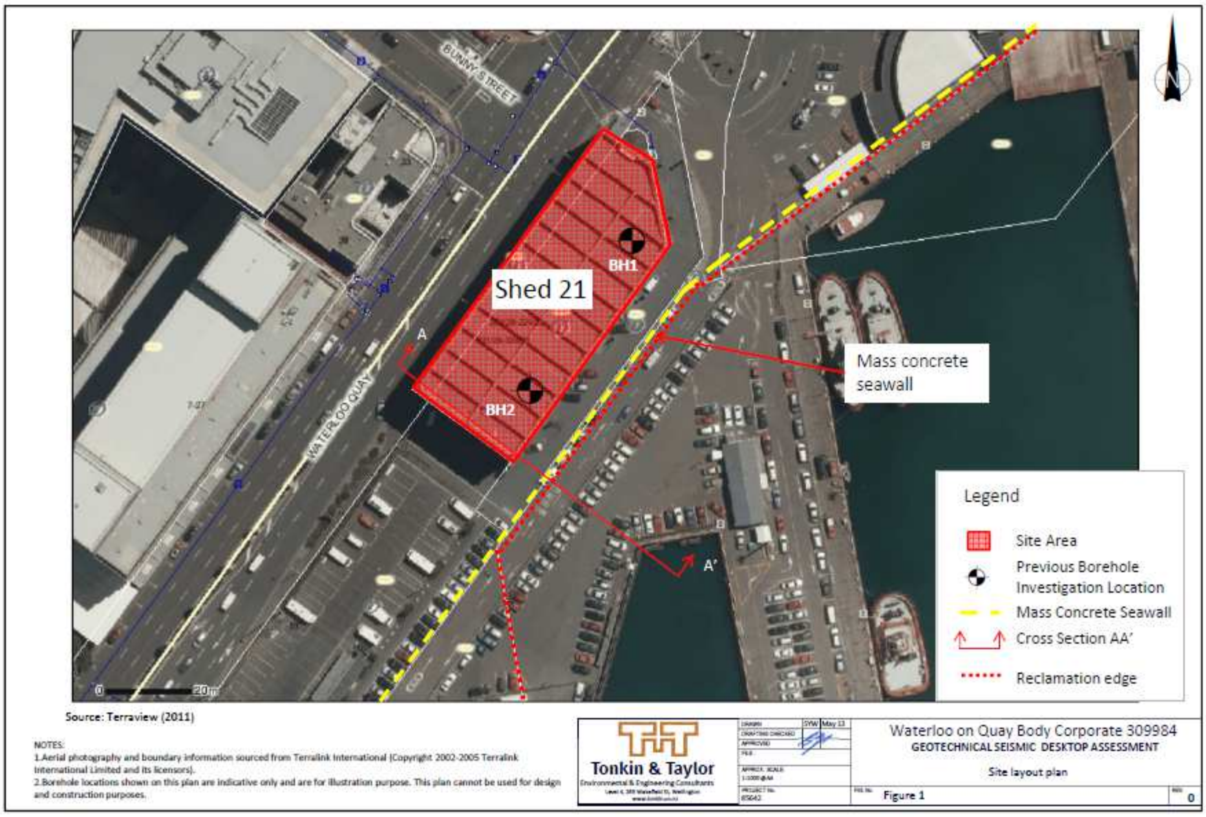
Yours sincerely

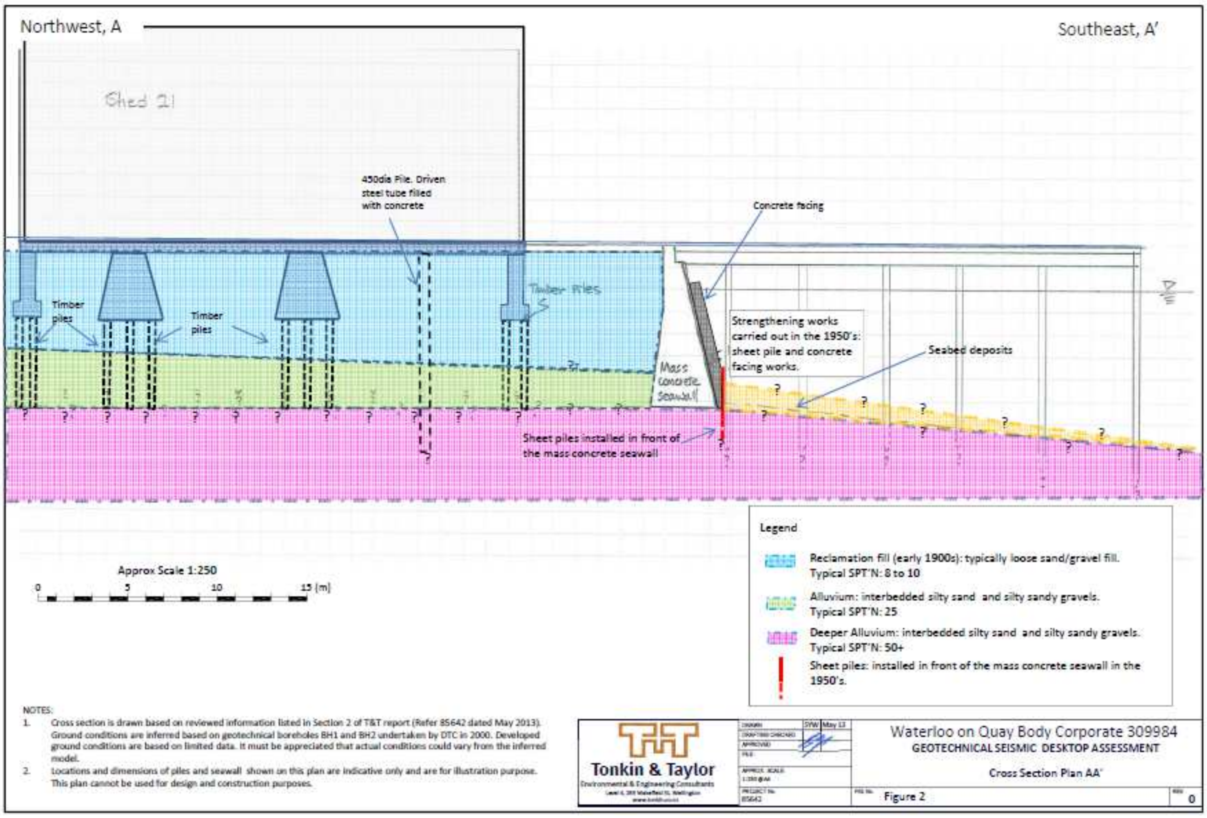
Stuart Palmer
Project Director

Attached: Figures 1 and 2

CC: Peter Johnstone

[illegible]


BUNNY STREET
WATERLOO QUAY
Shed 21
BH1
BH2
A
A'
Mass concrete seawall
Legend
Site Area
Previous Borehole Investigation Location
Mass Concrete Seawall
Cross Section AA'
Reclamation edge
0
20m
Source: Terraview (2011)
NOTES:
1.Aerial photography and boundary information sourced from Terralink International (Copyright 2002-2005 Terralink International Limited and its licensors).
2.Borehole locations shown on this plan are indicative only and are for illustration purpose. This plan cannot be used for design and construction purposes.
Tonkin & Taylor
Environmental & Engineering Consultants
SYW May 13
Waterloo on Quay Body Corporate 309984
GEOTECHNICAL SEISMIC DESKTOP ASSESSMENT
Site layout plan
Figure 1
0

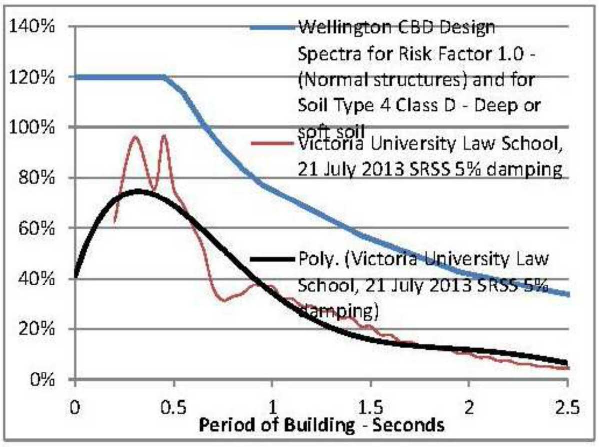

Northwest, A
Southeast, A'
Shed 21
450dia Pile. Driven steel tube filled with concrete
Concrete facing
Timber piles
Timber piles
Timber piles
Strengthening works carried out in the 1950's: sheet pile and concrete facing works.
Seabed deposits
Mass concrete seawall
Sheet piles installed in front of the mass concrete seawall
Legend
Reclamation fill (early 1900s): typically loose sand/gravel fill. Typical SPT'N: 8 to 10
Alluvium: interbedded silty sand and silty sandy gravels. Typical SPT'N: 25
Deeper Alluvium: interbedded silty sand and silty sandy gravels. Typical SPT'N: 50+
Sheet piles: installed in front of the mass concrete seawall in the 1950's.
Approx Scale 1:250
0
5
10
15 (m)
NOTES:
1. Cross section is drawn based on reviewed information listed in Section 2 of T&T report (Refer 85642 dated May 2013). Ground conditions are inferred based on geotechnical boreholes BH1 and BH2 undertaken by DTC in 2000. Developed ground conditions are based on limited data. It must be appreciated that actual conditions could vary from the inferred model.
2. Locations and dimensions of piles and seawall shown on this plan are indicative only and are for illustration purpose. This plan cannot be used for design and construction purposes.
Tonkin & Taylor
Environmental & Engineering Consultants
SYW May 13
Waterloo on Quay Body Corporate 309984
GEOTECHNICAL SEISMIC DESKTOP ASSESSMENT
Cross Section Plan AA'
Figure 2
0

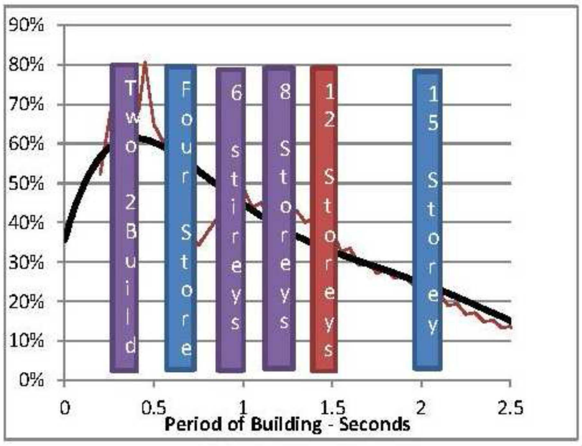
17. Appendix G – Comparative Size of the July 2013 Cook Strait Earthquake

This shaking that the building experienced was greater than the “moderate” earthquake referred to in the Earthquake Prone Building legislation. Preliminary data from the GNS strong motion instruments at the University of Victoria Law School (in the new lecture theatre on Stout Street) estimate the response spectra measured to be approximately 50 to 60% (say 55%) of New Building Standard (%NBS). See below for a four storey building.

% New Building Standard (NBS) - Comparison of Recorded Shaking with the Design Code Levels - Wellington CBD Design Spectra for Risk Factor 1.0 - (Normal structures) and for Soil Type 4 Class D - Deep or soft soil for Victoria University Law School, 21 July 2013 % New Building Standard (NBS)

Importance Levels	Normal structures		Importance Level 2
	Consequence of Failure - Ordinary		
	Importance Level Multiplier		1.0
	Description - Medium consequence for loss of human life, or considerable economic, social or environmental consequences.		
	Comment - Normal structures and structures not falling into other levels.		
Design Working Life	50 years		
Subsoil interpolation C trend D	75%		
Site Subsoil Class	4 Class D - Deep or soft soil		
Design Working Life	50 years		
Location in NZ	Wellington CBD		Z value 0.40
Building Material	Concrete		
Limit State Description	EQ ULS	SLS1	SLS2
Return period	500	25	N/A
Return period factor	1.00	0.25	#N/A
Strong Motion Site	Victoria University Law School, 21 July 2013		





Wellington CBD Design Spectra for Risk Factor 1.0 - (Normal structures) and for Soil Type 4 Class D - Deep or soft soil for Victoria University Law School, 21 July 2013 % New Building Standard (NBS)

DISCLOSURE STATEMENT

1. The following information has been supplied to Capital Commercial (2013) Limited ("Bayleys") to be made available for distribution on the vendor's behalf to potential purchasers to assist purchasers with their due diligence and to use at the purchaser's discretion.
2. Bayleys and the Vendor do not warrant the accuracy or completeness of the information and recommends that all recipients undertake their own due diligence, obtain their own reports to their satisfaction and seek independent advice prior to committing to purchaser.