DETAILED ENGINEERING EVALUATION – QUALITATIVE AND QUANTITATIVE REPORT

NEW BRIGHTON CLOCK TOWER

September 2012

Capital Programme Group

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NEW BRIGHTON CLOCK TOWER
(213 Marine Parade, New Brighton)

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EXECUTIVE SUMMARY

Structural inspections of the clock tower undertaken follow the 4th of September 2010, 22th of February earthquakes and subsequent aftershocks revealed minor new cracking and the opening of existing damage which has not compromised its load resisting capacity.

The building has been assessed initially using the NZSEE Initial Evaluation Procedure (IEP), and has been assessed to be 25% New Building Standard (NBS) and therefore a full detailed engineering evaluation (DEE) has been undertaken.

The Detail Engineering Evaluation indicates the structure to have a seismic capacity of 58% NBS. It is therefore not considered Earthquake Prone. The building is therefore considered a moderate risk and can remain in use.

This report recommends that the structure requires maintenance to reduce the corrosion of the concrete reinforcement and spalling of the concrete surface in some locations.

If the client desires to upgrade the structure to 100% of the New Building Standard, further geotechnical investigations and complex analytical modelling will be required.

Strengthening of the building is likely to involve intrusive work to the foundations and superstructure which may need to be given careful considerations due to the structure's heritage status.

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2. BACKGROUND

Capital Programme group has been engaged by the Transport and Green space Unit to undertake a detailed engineering evaluation of the New Brighton clock tower.

This report comprises two type of analysis: the first analysis is the initial Qualitative Assessment of the building structure based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. The second is a detail assessment based on the rocking mechanism of the structure.

A qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available. The detail assessment is a more realistic approach based actual material testing, load distribution analysis and computer modelling to represent the actual building condition.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

3. COMPLIANCE

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at time of writing.

3.1. CANTERBURY EARTHQUAKE RECOVERY AUTHORITY

CERA was established on 28th March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18th April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

3.2 BUILDING ACT

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority, i.e. the Christchurch City Council (CCC) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy
This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

3.3 CHRISTCHURCH CITY COUNCIL POLICY

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1st July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above. The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

3.4 BUILDING CODE

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

4. EARTHQUAKE RESISTANCE STANDARDS

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis. The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 4a below. Figure 4b (extracted from AISPBE) compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	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	Unacceptable	Unacceptable

Figure 4a NZSEE Risk Classifications Extracted from Table 2.2 of the NZSEE 2006 AISPBE

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

Figure 4b %NBS compared to the relative risk of failure.(Refer Table 2.1 of the NZSEE 2006 AISPBE)

5. BUILDING DESCRIPTION

5.1 GENERAL

The clock tower consists on 3 storey reinforced concrete building with concrete ribbon cladding structural walls from the base to approximately mid height of the structure with a total height of 12.1m height and square shape of 3.55m long with a floor foot print area of 12.6m2 and was built circa 1934.

The gravity and lateral resistance is provided by two long resisting walls in the East-West direction while four short walls connected by a archway are provided in the North-South direction. Copies of the architectural plan are shown in Appendix C of this report. These drawings were provided following a building survey as part of the detail engineering evaluation process.

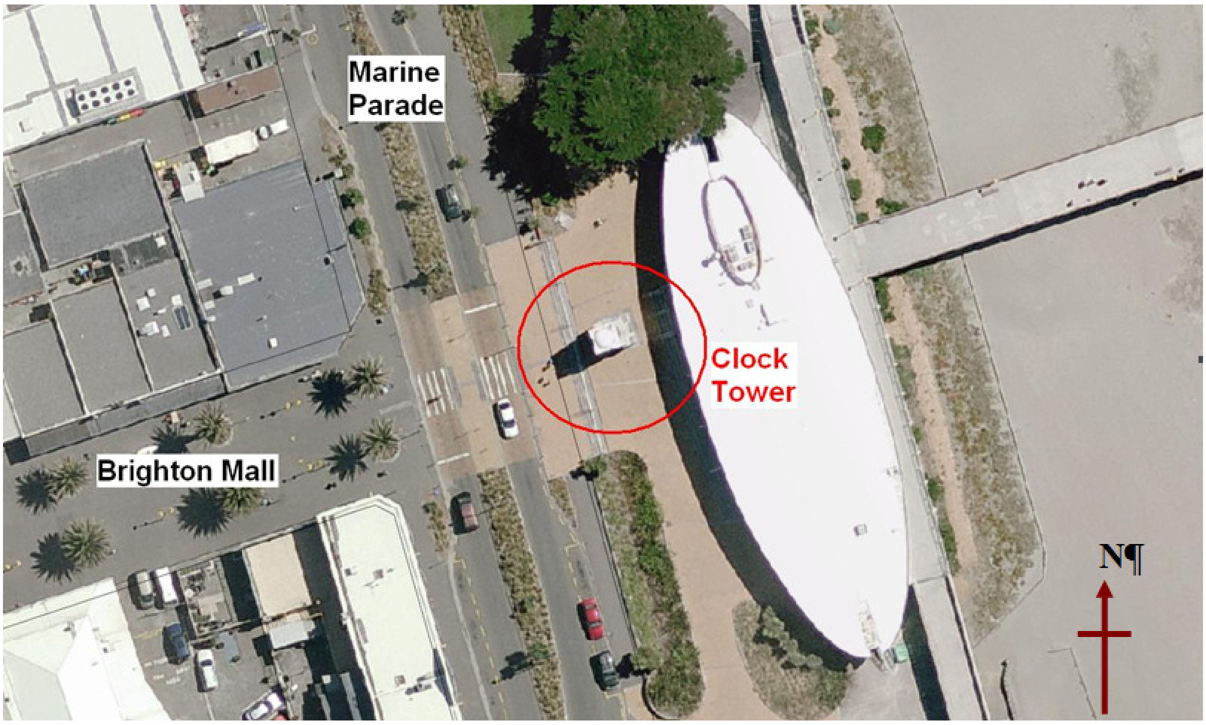


Figure 5a – Plan view showing location of clock tower

5.2 GRAVITY AND SEISMIC LOAD RESISTING SYSTEM

The gravity loads acting on the structure correspond to the self weight of the concrete wall and floor elements in addition to the ribbon cladding walls.

Gravity loads from the reinforced concrete roof, second and first floor slabs are transferred through a monolithic connection between the slab and the walls. The loads are then transferred into a reinforced concrete foundation. It should be noted that the foundations for the building were not inspected. It was not possible to find any existing structural drawings for the structure. This report, however assumes that foundation dimension are similar to the dimension at the base of the walls (300mm wide) which seems reasonable and conservative assumption.

Lateral loads acting on the structure are resisted by the reinforced concrete walls in all directions of the building and then transferred into the foundations. During an earthquake the building is expected to behave in a relatively stiff manner due to the size and arrangement of walls in both directions and the connection of the floor slabs. The geometric nature of the structure is likely to initiate a rocking mechanism under dynamic loading conditions above a certain threshold which will help dissipate the seismic energy.

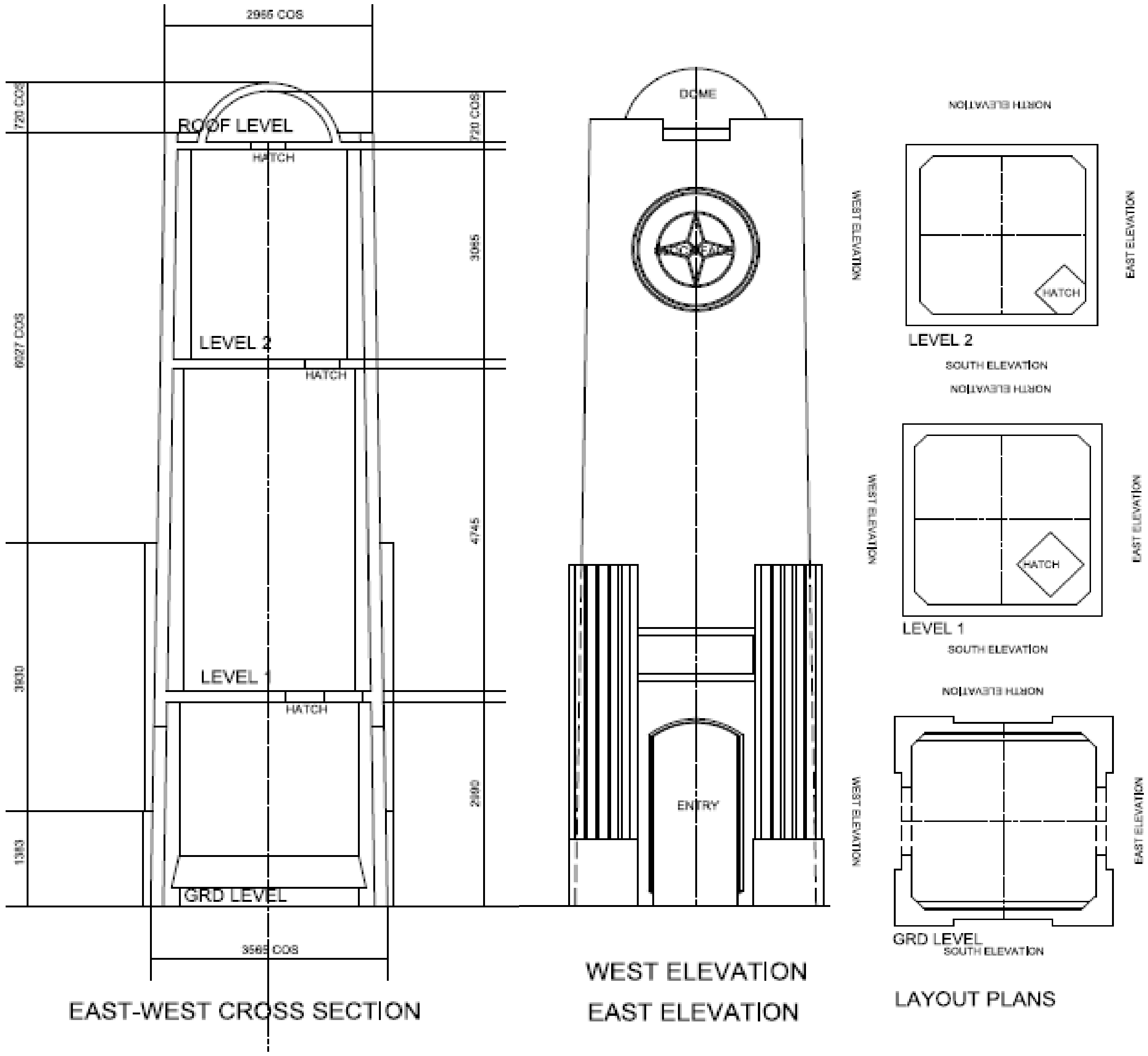


Figure 5b – Elevation and plan view of the structure

6. ASSESSMENT

An inspection of the building was undertaken on the 25th of March 2011 and subsequent inspections following a major aftershocks. The latest inspection compressed a full external and internal inspection and was carried out on the 5th of April 2012.

Inspections did not reveal a civil defence placard on the building, and investigations with the Christchurch City Council revealed no record of the placard of this structure either. The building was assumed to have a green placard in place as no restriction on acess were enforced. The clock tower operator had mentioned that the clock stopped operating after the quakes and access to fix the clock was not prohibited.

A large portion of the main structural components of the building were able to be viewed since they are uncovered. The concrete walls are unlined internal and externally. No inspection of the foundations of the structure was undertaken. Reinforcing bars within the concrete walls were scanned to find their spacing and concrete strength testing was undertaken using a Schmidt Hammer.

The site was assessed for damage, including observing the ground conditions, checking for damage in areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in structural and non- structural elements.

The %NBS score determined for the building is based on the IEP procedure described by the New Zealand Society of Earthquake Engineering (NZSEE) from information obtained by visual observation of the building only as no record drawings are available.

7. DAMAGE ASSESSMENT

General observations are as follows:

- The building condition is not considered to pose an immediate risk to the public.
- There is minor cracking throughout the structure which is most likely due to a combination of old, poor quality concrete and the effects of the earthquake. Overall the cracking has likely worsened and there is some new cracking which can be attributed to the earthquakes.
- The tower appears to be vertical.
- From ground level there appears to be a horizontal and vertical cracks (and some diagonal cracks) on the upper western face (Figure 7a). These cracks were noted in a two reports dated in February 1993 and October 2009 and so are therefore pre-earthquake (but may have worsened since). The horizontal crack corresponds to a poor quality of concrete in a construction joint (two stages of concrete pouring while construction). A closer inspection of the crack with did not show signal of propagation from East to West (note the crack was not visible on the internal face). The width of the crack was measured to be 0.4mm. note that from ground level the

crack appear wose (thicker) due to a build-up of dirt and mold behid the paint at the crack locations.

Figure 7a – Horizontal crack on the west face

- A horizontal crack was observed in the side of the penetration through the level 2 floor slab (Figure 7b)

Figure 7b – Interior of the north and East faces showing the vertical cracks and moisture inside the clock tower

- Beside one of the clock faces there is a vertical crack (figure 7c).

Figure 7c – Vertical crack found on the side of one of the clock faces: location (left). Close up (right)

- Separation between the mortar and concrete on a few of the clock faces. (figure 7d)

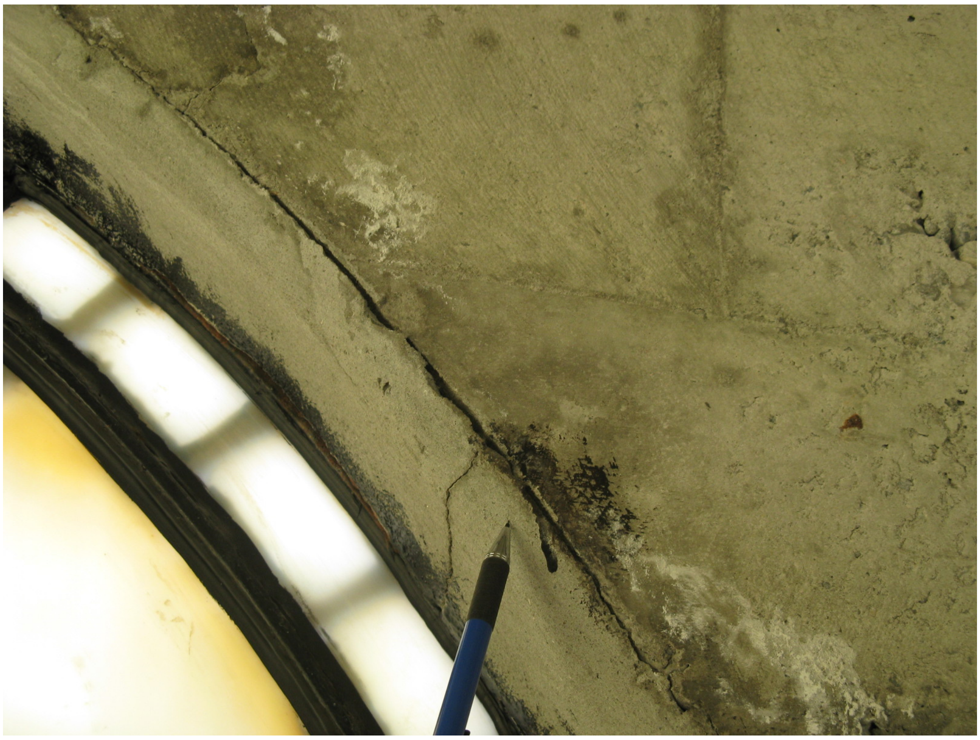


Figure 7d Separation between mortar and concrete

- Corrosion coming from the clock framing in the East face (Figure 7e)

Figure 7e Rusting stain on East face

8. CRITICAL STRUCTURAL WEAKNESS

8.1 SHORT COLUMNS

No short columns were observed in the building.

8.2 PLAN AND VERTICAL IRREGULARITY

The building is square with no irregularities in plan or height and therefore for the purposes of the IEP assessment of the building, and determination of the %NBS score, no reduction has been assessed in accordance with the NZSEE guidelines.

8.3 STAIRCASE

There is no staircase in the structure. Access to the first and second floors is provided by ladder.

8.4 ROOF DAMAGE

No critical structural weaknesses were identified in the roof structure.

9. GEOTECHNICAL CONSIDERATION

9.1 SITE DESCRIPTION AND GEOTECHNICAL INVESTIGATION

The site is relatively flat and is situated adjacent to the New Bridgton beach elevated approximately two meters above mean sea level. No liquefaction was observed from the post 22nd February 2011 Aerial survey (Fig 9a).

Information from Environment Canterbury (ECan) indicates one borelog in close proximity to the clock tower. Classified as N35/2047, the borelog was taken in 1906 to a depth of 90m from ground level. The site geology described indicates sand down to 20m and follow by layer of clay and sand.

Fig 9a. Location of Ecan borelog investigated

It should be noted that the purpose of the boreholes and well logs from Ecan were for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording is variable and may not be representative. Soil strength data was not recorded.

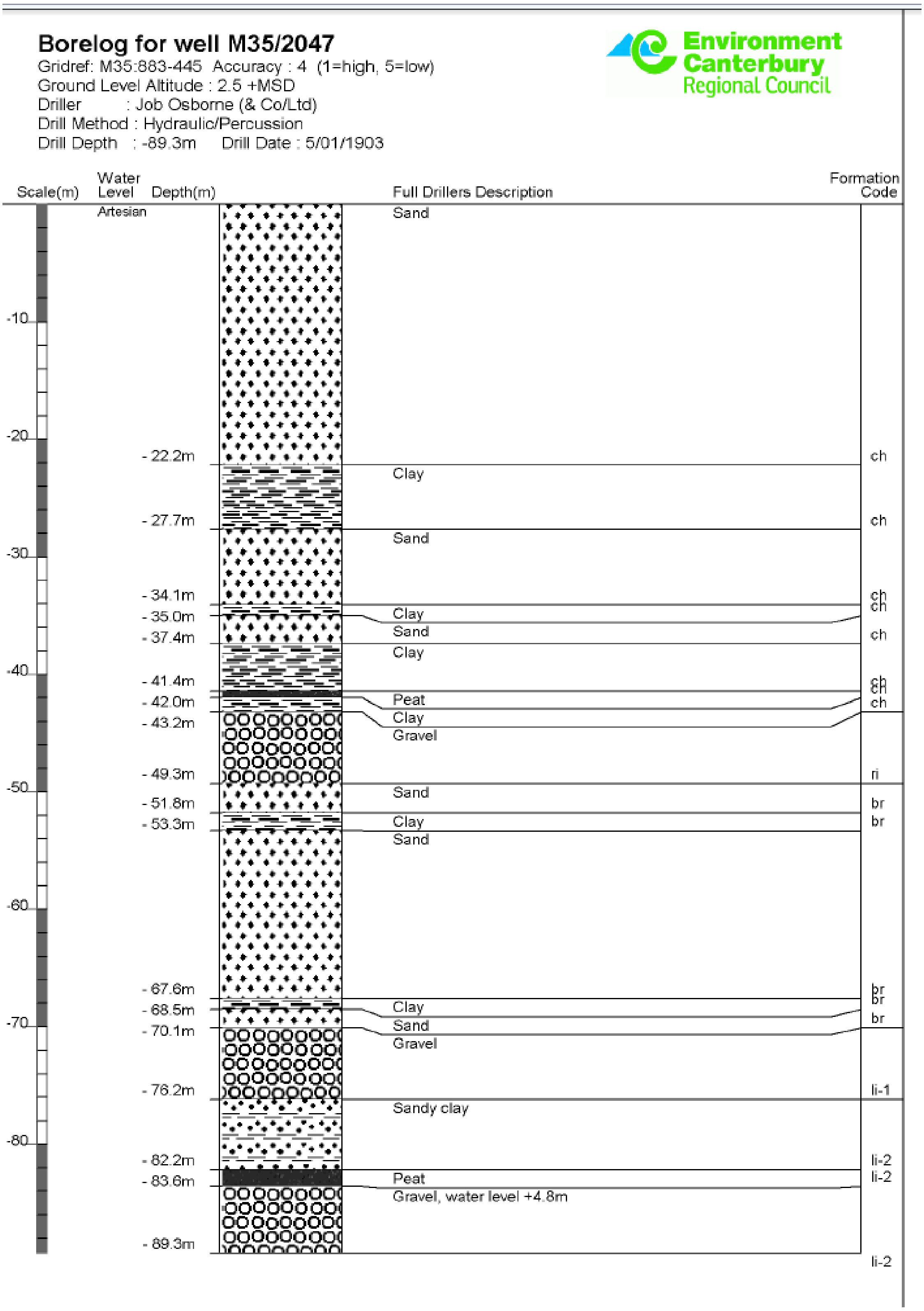


Fig 9a. Ecan borelog lothology

This assessment is based on a desktop review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site appears to be situated on hard sand deposits, and potentially gravel at 40m depth. The site has been identified as having moderate liquefaction potential, in particular where sands and/or silts are present. Isolated lithologies may be susceptible to liquefaction; however this is not anticipated to have significant detrimental effects on structures and amenities at the ground surface. It should be noted that little or no liquefaction occurred at or around the site following the major earthquakes.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that an intrusive investigation comprising of a minimum of two piezocone CPT be conducted. A soil class of D (in accordance with NZS 1170.5:2004) has been adopted for the site for the purposes of the detail analysis.

9.2 SLOPE FAILURE AND ROCKFALL POTENTIAL

The site is located well away from slope or rockfall issues.

10. SURVEY

No level or verticality surveys were conducted for this building. However, the tower appears to be vertical. There is no evidence of lateral movement of the tower, however, concrete block paving around the structure may allow for movement of the structure whithout showing signs of damage.

11. INITIAL STRUCTURAL CAPACITY ASSESSMENT

11.1 %NBS ASSESSMENT

The building's capacity has been assessed using the Initial Evaluation Procedure based on the information available. The building's capacity excluding critical structural weaknesses, and the capacity of any identified weaknesses are expressed as a percentage of New Building Standard (%NBS) are shown in Table 11a.

Table 11a Indicative Building and Critical Structural Weaknesses Capacities based on the NZSEE Initial Evaluation Procedure (IEP)

Item	%NBS
Building excluding CSW's	25

The building is therefore considered potentially Earthquake Prone as it achieves less than 33% NBS. This score has not been adjusted when considering damage to the structure as all damage observed was relatively minor and considered unlikely to adversely affect the load carrying capacity of the structural systems.

11.2 DISCUSSION OF RESULTS

The IEP score is generally considered as a guide for the buildings expected performance in a seismic event and is based upon general parameters.

The results obtained from the initial IEP assessment are consistent with those expected for a building of this age and construction type founded on Class D soils. The original building was constructed circa 1934 and there was no earthquake design loading standard current at the time (other than for buildings constructed in Wellington).

Since only minor damage was observed and the structure appears to have performed much better than the IEP score would suggest a detailed capacity assessment was undertaken to establish a more refined score.

12. DETAIL CAPACITY ASSESSMENT

12.1 MATERIAL PROPERTIES AND MEMBER STRENGTHS

An assessment of the material concrete strength was evaluated on site using a Schimitt hammer rebounding test. This measures the hardness of the concrete surface and can be calibrated to provide an estimate of the concrete strength.

A representative number of tests were performed at the ground level. It was found a good f'c=30 MPa can be used conservatively, which seems reasonable. However, maintenance report dated back to 1993 identified concerns with the state of the concrete which it was identified with very poor quality in some areas and the majority of reinforced concrete bars have rusted out. Specific concerns were at the construction joint at mid way up the central chamber (where the reduction of cross section area is). Therefore, this report assumes f'c=17 MPa for calculation purposes.

Non intrusive location and depth testing was undertaken for the steel reinforcement using a HILTI –bar scanner. This determined that one layer of longitudinal reinforcing at 350mm spacing and 350mm for the vertical reinforcing bars was within the concrete walls. The scan indicated the depth of the bars but did not provide the bar diameters. It was assumed round R12 were used. The NZTA manual section 5.1 suggests that for this age of structure a tensile strength of 210 MPa can be used for the steel reinforcing. Figure 12.1 shows a plan cross section and the steel bar layout assumed within the walls, based on the site measurements.

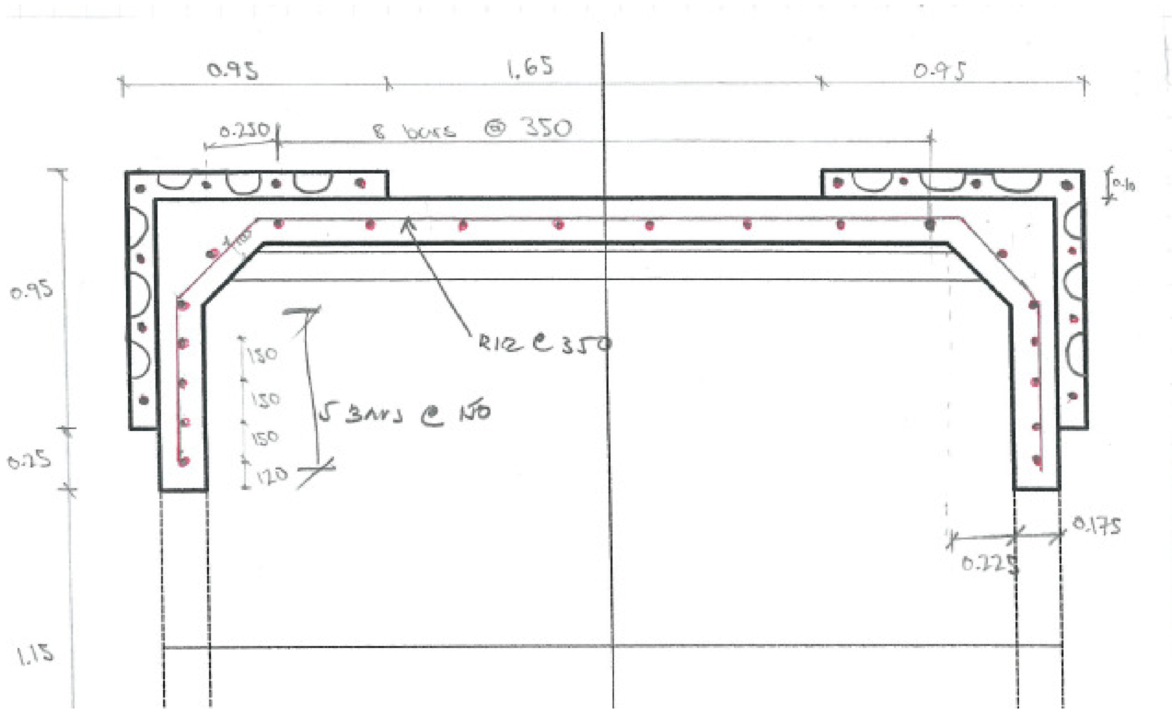


Fig 12. Cross section of walls and reinforcement

12.2 SEISMIC PARAMETERS

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil (assumed)
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011. The increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing

recommendations (effectively resulting in a reduced % NBS score compared to IEP's undertaken prior to that date).

- Return period factor Ru = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life (note that the clock tower was constructed 78 years ago).
- A structural ductility factor of 1.0 has been assumed in both the long and short direction at both Serviceability limit state (SLS) and ultimate limit state (ULS) of the building based on engineering judgment (construction and detailing age of the structure). This ductility factor can be increased to 1.25 to make the study less conservative.
- The seismic coefficient was calculated as 0.21g and 0.63g for SLS and ULS respectively. This equates that total mass participating in the inertial forces during an earthquake.

12.3 LOADS

Total weight of the structure was calculated as 578kN including concrete floor. No live load was accounted in the tower as no access to the tower is allowed, and there is a negligible load due to the clock mechanism.

The total seismic weight was calculated as 728kN. It is assumed that two long walls resist the seismic forces in the East-West direction (364 kN total shear for each long wall) and four short walls in the North-South direction (182 kN per short wall).

Mass redistribution for the first floor was calculated assuming that the mass of the first floor is the contribution (based on half of the first and second floor height), while the mass contribution of the second and third floors were based on half of the first and second floor heights respectively. Total floor weight was calculated as 184.8 kN, 210.2 kN and 33 kN for the first, second and third floor respectively.

12.4 STRUCTURAL ANALYSIS METHOD

Equivalent static method analysis was used to obtain the lateral load distribution according to AS/NZ 1170.5 Earthquake Actions. Total base shear demand at ULS was estimated to be 182 kN and overturning moment of 1415 kNm at the base of the structure for the long walls direction. For the short walls, the total base shear was 91 kN and overturning moment of 708 kNm. Table 12.a shows the lateral loads applied at each floor.

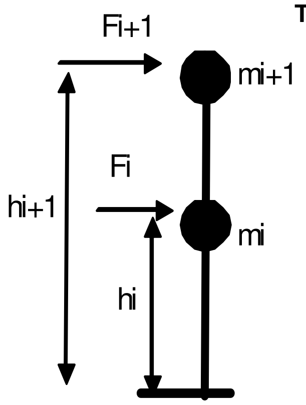


Table 12a Equivalent static method Procedure

				LONG WALL DIRECTION			
				SLS		ULS	
LEVEL	mi (kN/frame)	Hi (m)	miHi (kNm)	Fi (kN)	FiHi(kNm)	Fi(kN)	FiHi(kNm)
3	16.5	11.30	185.885	13	142	38	427
2	105.1	8.12	853.006	36	289	107	867
1	92.4	3.23	298.452	12	40	37	121
TOTAL			1337	61	472	**182**	**1415**

				SHORT WALL DIRECTION			
				SLS		ULS	
LEVEL	mi (kN/frame)	Hi (m)	miHi (kNm)	Fi (kN)	FiHi(kNm)	Fi(kN)	FiHi(kNm)
3	8.2	11.30	92.9425	13	142	19	214
2	52.5	8.12	426.503	36	289	53	434
1	46.2	3.23	149.226	12	40	19	60
TOTAL			669	61	472	**91**	**708**

12.5 STRUCTURAL ASSESSMENT METHOD

An initial section capacity of the structural components assuming fixed foundation determined that the walls in both direction do not have sufficient capacity to resist the seismic forces generated by lateral load at 100% NBS. In particular the tension forces in the walls due to the high overturning moments were in excess of the wall capacity.

However, since there was no evidence that the walls had been overstressed, it suggests that the dynamic response of the clock tower was a rocking motion subjected to uplifting. However, there was no presence of cracking around the concrete block paver, but this can accommodate some movements.

Rocking motion is a commonly observed phenomenon with complex and nonlinear behaviour. It can be shown that by allowing rocking to take place in structures, the resulting accelerations and hence forces can be significantly reduced. This approach when applied correctly acts as an effective isolation mechanism for structures against severe ground motion. A rocking structure normally enhances the seismic resistance and their post-earthquake serviceability such as in this case.

A prevalent sentiment amongst practitioners is that although the philosophy behind rocking as a seismic isolation solution is logical, it is prudent not to implement it for high seismic areas until the system performance has been tested by an actual major earthquake in addition to the lack of available guidelines for engineers.

The assessment of the clock tower's capacity has been undertaken in accordance with a paper recently published by the Structural Engineering Society new Zealand in April 2011 "Tentative Seismic Design guidelines for Rocking Structures" By Trevor Kelly.

This paper predicts the rocking uplifting structures by defining a foundation size and calculates spring soil stiffness given the shear modulus and Poisson's ratio from the soil characteristics and assesses the wall rocking strength, seismic displacements, drift and general performance of the walls.

The analysis presented in the paper is applicable for walls with relatively small ductility factors (DF) with a rocking strength (static restoring moment) of the one-quarter or more of the elastic demand (i.e. DF<4.0) which is applicable to this structure.

12.6 %NBS DETAIL ASSESSMENT

An assumed foundation width of 0.3m for the long wall and 1.8m (this is taking into consideration the continuity of the wall acting as a T section) for the short wall directions were used to assess the capacity of the rocking structure. Soil properties were evaluated by a parametric analysis using upper, medium and lower values of dense to medium sand. Shear modulus, Poisson's ratio soil failure stress and ultimate capacities where are given in Table 12.b.

Table 12b Range of soil properties analyzed

Soil	**Type**	**Shear Modulus** (kPa)	**Posson's Ratio**	**ULS (kPa)**
Dense Sand	Upper	80000	0.4	650
Medium Sand	medium	60000	0.35	550
Soft Sand	Low	40000	0.3	450

The building had been evaluated by top drift and shear and are found to be within the drift limit and capacity of the concrete wall with the reinforcement. Table 12c shows the results and compared with the current standards.

It can be seen that as the structure is allowed to rock (i.e increase the drift), the total shear demand decreases and the structure is stiffer in the long walls direction than the short walls.

In terms of drift demand the structure complies with the drift limit in the long wall direction. However, for the short wall, the drift capacity is between 58% and 71% of the new building standard (NBS).

Similarly, in terms of shear wall capacities, the walls are well in excess of the shear demand generated by the uplifting. However, the spacing of horizontal and vertical shear reinforcement, and the horizontal shear reinforcement in the walls and the ratio of vertical reinforcement area to gross concrete are of horizontal section do not comply with the NZ3101:2006 Concrete Structure Standards.

Table 12c Indicative Building capacity based on the DEE

LONG WALL				
Soil	**Top drift (%)**	**%NBS** (2.5% limit)	**V* (kN)**	**ϕVn (kN)**
Dense Sand	1.2	208	59	343
Medium Sand	1.6	156	55	343
Soft Sand	2.2	113	49	343
SHORT WALL				
Dense Sand	3.5	71	19	168
Medium Sand	3.9	64	18	168
Soft Sand	4.3	58	15	168

Medium dense sand with ultimate soil capacity of 550 Mpa, with shear modulus of 60000 kPa and passion's ratio of 0.35 is an average value for this assessment and represents the actual conditions for the structure. Therefore, with this assumption, the structure will possess a minimum of 64% NBS.

13 CONCLUSIONS

The damage to the structure during recent seismic activity in Christchurch has caused minor crack damage and open existing damage due to ageing. The building suffered insignificant damage that would not compromise the load resisting capacity of the existing structural systems.

The building is therefore not considered potentially Earthquake Prone as it achieves more than 58% NBS. This score is based on assumed foundation size and geotechnical parameters based on a near by borehole log and seems to be appropriated when considering damage to the structure as all damage observed was relatively minor and considered unlikely to adversely affect the load carrying capacity of the structural systems. As a result it is recommended that the building remain in use.

Minor repairs are required to prevent corrosion on the reinforcing steel and concrete spalling.

14 RECOMMENDATIONS

This report recommend as follows:

- The structure requires maintenance to reduce the corrosion of the bars and spalling of the concrete in some parts of the structure.
- To upgrade the structure to a higher level of the new building standard, strengthening of the foundations by using screw piles or additional concrete foundations in combination of strengthening the walls using FRP or similar products would be an option. This will require geotechnical investigations and model complex computer analysis of the structure.
- Repair the cracks by using a suitable crack injection such a SIKA InjectoKit TH. Interior cracks should also be addressed with the crack injection method. A full survey and methodology should be undertaken to address the concrete crack repair method required.

15. LIMITATIONS

15.1 GENERAL

This report has been prepared subject to the following limitations and assumptions:

- Consented drawings of the building were not available.
- The information contained in this report has been obtained from visual inspections of the building, non destructive testing and available literature review of model analysis.
- The foundations of the building were not inspected.
- No intrusive structural investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.

It is noted that this report has been prepared at the request of the Client and is intended to be used for their purposes only. The author accepts no responsibility for any other party or person who relies on the information contained in this report.

15.2 GEOTECHNICAL LIMITATIONS

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of the Client and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. The author accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal and limited desktop study. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by the author. The author accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

16 APPENDIX A INITIAL EVALUATION PROCEDURE

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	New Brighton Clock Tower	Reviewer:	
	Unit / No: / Street	CPEng No:	
Building Address:		Company:	CCC
Legal Description:		Company project number:	
		Company phone number:	
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	05/03/2012
		Revision:	
Building Unique Identifier (CCC):		Is there a full report with this summary?	

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	3	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	12.20		height from ground to level of uppermost seismic mass (for IEP only) (m):	12.2
Floor footprint area (approx):	13			
Age of Building (years):	80		Date of design:	Pre 1935
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):	other (specify)			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	concrete	slab thickness (mm)	125
Floors:	concrete flat slab	slab thickness (mm)	125
Beams:	none	overall depth x width (mm x mm)	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Lateral system along:	concrete shear wall	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	3.9
Ductility assumed, μ:	1.00		wall thickness (m):	0.175
Period along:	0.37	1.37 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	non-ductile concrete moment frame		note typical bay length (m)	3.6
Ductility assumed, μ:	1.00			
Period across:	0.41	0.00	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:	other (specify)	describe	
Wall cladding:			
Roof Cladding:			
Glazing:			
Ceilings:			
Services(list):			

Available documentation

Architectural	partial	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:		notes (if applicable):	
Differential settlement:		notes (if applicable):	
Liquefaction:		notes (if applicable):	
Lateral Spread:		notes (if applicable):	
Differential lateral spread:		notes (if applicable):	
Ground cracks:		notes (if applicable):	
Damage to area:		notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:		Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:			
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS(before) - \%NBS(after))}{\%NBS(before)}$$

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	
Along	Assessed %NBS before:		24% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:			
Across	Assessed %NBS before:		24% %NBS from IEP below	
	Assessed %NBS after:			

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): Pre 1935

h_n from above: 12.2m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.37	0.41
(%NBS)nom from Fig 3.3:	5.0%	5.0%

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final $(\%NBS)_{nom}$:	5%	5%

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		1.00
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	0.30
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B**:	3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	2.00	2.00
Ductility scaling factor: =1 from 1976 onwards; or $=k_\mu$, if pre-1976, fromTable 3.3:	1.00	1.00
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

	along	across
$\%NBS_b$:	17%	17%

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential	Pounding effect D1, from Table to right	1.0
	Height Difference effect D2, from Table to right	1.0
	Therefore, Factor D:	1
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.5	1.5
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

	Along	Across
3.7. Overall Performance Achievement ratio (PAR)	1.50	1.50
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	25%	25%
4.4 Percentage New Building Standard (%NBS), (before)		25%

fficial Use only:

Accepted By:
Date:

17 APPENDIX B SPREADSHEET CALCULATIONS

Long wall hard soil

LONG WALL GEOMETRY									
H (m)	11.30	Co	1.2	B (m)	0.13	Ve (kN)	54		
L (m)	3.55	V_E (kN)	182	Selected	0.3				
W_D (kN)	289	V_R (kN)	32	DF	5.7		T upper limit (sec)		0.258
Qc (kPa)	650	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.050		G =	80000	Kend	910667			
L2 (m)	0.575		v =	0.4	Kmid	97333			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.025	-1.750	45533	31	1	139446			
2	0.338	-1.438	55967	38	13	115650			
3	0.913	-0.863	55967	38	35	41634			
4	1.488	-0.288	55967	38	56	4626			
5	2.063	0.288	55967	38	78	4626			
6	2.638	0.863	55967	38	100	41634			
7	3.213	1.438	55967	38	122	115650			
8	3.525	1.750	45533	31	109	139446			
		0	426867	289	513	602711			

COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH			
c (m)	1.48	Cm	1.0	Fy (kN)	32	Cy	0.110

	UPPER BOUND				
	Ti	C(Ti)	Re	Te	Te/Ti
1	0.258	0.630	5.738	1.482	5.738
2	0.870	0.452	4.114	1.063	1.221
3	0.966	0.417	3.802	0.982	1.016
4	0.974	0.415	3.778	0.976	1.002
5	0.975	0.415	3.775	0.975	1.000
6	0.975	0.415	3.775	0.975	1.000
7	0.975	0.415	3.775	0.975	1.000
8	0.975	0.415	3.775	0.975	1.000
9	0.975	0.415	3.775	0.975	1.000

SEISMIC DISPLACEMENT		TOP WALL DISPLACEMENT	
ϖ upper bound (mm)	98	ϖ upper bound (mm)	118
ϖ lower bound (mm)	0	ϖ lower bound (mm)	0

DUCTILITY FACTOR		DYNAMIC AMPLIFICATION EFFECT					
DF =	5.7	Avn	0.15	min limit	1.86	max lim	3.5

TORSIONAL INCREASE IN DISPLACEMENT

not including in this spreadsheet

ASSESSMENT OF PERFORMANCE							
$ϖ_{kdm}$	1.2	drift upper limit(%)	1.2	OK	drift lower limit (%)	0.0	OK

Therefore the foundation width of 0.3 is satisfactory provided the walls design shears are satisfactory

Vx (kN)	59	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag	69	Mpa

PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT

Long wall medium soil

LONG WALL GEOMETRY									
H (m)	11.30	Co	1.2	B (m)	0.15	Ve (kN)	54		
L (m)	3.55	V_E (kN)	182	Selected	0.3				
W_D (kN)	289	V_R (kN)	28	DF	6.6		T upper limit (sec)		0.310
Qc (kPa)	550	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.050		G =	60000	Kend	630462			
L2 (m)	0.575		v =	0.35	Kmid	67385			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.025	-1.750	31523	31	1	96539			
2	0.338	-1.438	38746	38	13	80065			
3	0.913	-0.863	38746	38	35	28824			
4	1.488	-0.288	38746	38	56	3203			
5	2.063	0.288	38746	38	78	3203			
6	2.638	0.863	38746	38	100	28824			
7	3.213	1.438	38746	38	122	80065			
8	3.525	1.750	31523	31	109	96539			
		0	295523	289	513	417262			

COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH			
c (m)	1.75	Cm	1.0	Fy (kN)	28	Cy	0.095

	UPPER BOUND				
	Ti	C(Ti)	Re	Te	Te/Ti
1	0.310	0.630	6.597	2.048	6.597
2	1.179	0.368	3.854	1.196	1.015
3	1.188	0.366	3.835	1.190	1.002
4	1.189	0.366	3.832	1.189	1.000
5	1.189	0.366	3.831	1.189	1.000
6	1.189	0.366	3.831	1.189	1.000
7	1.189	0.366	3.831	1.189	1.000
8	1.189	0.366	3.831	1.189	1.000
9	1.189	0.366	3.831	1.189	1.000

SEISMIC DISPLACEMENT		TOP WALL DISPLACEMENT	
4 upper bound (mm)	129	4 upper bound (mm)	154
4 lower bound (mm)	0	4 lower bound (mm)	0

DUCTILITY FACTOR		DYNAMIC AMPLIFICATION EFFECT					
DF =	6.6	Avn	0.15	min limit	1.99	max lim	3.5

TORSIONAL INCREASE IN DISPLACEMENT

not including in this spreadsheet

ASSESSMENT OF PERFORMANCE							
4_{kdm}	1.2	drift upper limit(%)	1.6	OK	drift lower limit (%)	0.0	OK

Therefore the foundation width of 0.3 is satisfactory provided the walls design shears are satisfactory

Vx (kN)	55	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag	64	Mpa

PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT

Long wall soft soil

LONG WALL GEOMETRY							
H (m)	11.30	Co	1.2	B (m)	0.18	Ve (kN)	54
L (m)	3.55	V_E (kN)	182	Selected	0.3		
W_D (kN)	289	V_R (kN)	22	DF	8.4	T upper limit (sec)	0.395
Qc (kPa)	450	wall thickness		0.3			
srping No.	8		UPPER BOUND				
L1 (m)	0.050		G =	40000	Kend	390286	
L2 (m)	0.575		v =	0.3	Kmid	41714	

node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi\text{-}x)^2$
1	0.025	-1.750	19514	31	1	59763
2	0.338	-1.438	23986	38	13	49564
3	0.913	-0.863	23986	38	35	17843
4	1.488	-0.288	23986	38	56	1983
5	2.063	0.288	23986	38	78	1983
6	2.638	0.863	23986	38	100	17843
7	3.213	1.438	23986	38	122	49564
8	3.525	1.750	19514	31	109	59763
		0	182943	289	513	258305

COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH			
c (m)	2.14	Cm	1.0	Fy (kN)	22	Cy	0.075

	Ti	C(Ti)	Re	Te	Te/Ti
		UPPER BOUND			
1	0.395	0.630	8.419	3.322	8.419
2	1.858	0.246	3.287	1.297	0.698
3	1.578	0.288	3.850	1.519	0.963
4	1.548	0.292	3.908	1.542	0.996
5	1.545	0.293	3.915	1.544	1.000
6	1.545	0.293	3.915	1.545	1.000
7	1.545	0.293	3.915	1.545	1.000
8	1.545	0.293	3.915	1.545	1.000
9	1.545	0.293	3.915	1.545	1.000

SEISMIC DISPLACEMENT		TOP WALL DISPLACEMENT	
upper bound (mm)	174	upper bound (mm)	208
lower bound (mm)	0	lower bound (mm)	0

DUCTILITY FACTOR		DYNAMIC AMPLIFICATION EFFECT					
DF =	8.4	Avn	0.15	min limit	2.26	max lim	3.5

TORSIONAL INCREASE IN DISPLACEMENT

not including in this spreadsheet

ASSESSMENT OF PERFORMANCE							
kdm	1.2	drift upper limit(%)	2.2	OK	drift lower limit (%)	0.0	OK
Therefore the foundation width of		0.3	is satisfactory provided the walls design shears are satisfactory				
Vx (kN)	49	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag				57	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT							

Short wall hard soil

WALL GEOMETRY							
H (m)	11.30	Co	1.2	B (m)	0.23	Ve (kN)	7
L (m)	0.95	V_E (kN)	91	Selected	1.8	T upper limit (sec)	0.588
W_D (kN)	144	V_R (kN)	6	DF	14.4		
Qc (kPa)	650	wall thickness		0.3			
srping No.	8		UPPER BOUND				
L1 (m)	0.300		G =	80000	Kend	910667	
L2 (m)	0.058		v =	0.4	Kmid	97333	

node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi\text{-}x)^2$
1	0.150	-0.325	273200	68	10	28857
2	0.329	-0.146	5678	1	0	121
3	0.388	-0.088	5678	1	1	43
4	0.446	-0.029	5678	1	1	5
5	0.504	0.029	5678	1	1	5
6	0.563	0.088	5678	1	1	43
7	0.621	0.146	5678	1	1	121
8	0.800	0.325	273200	68	54	28857
		0	580467	144	69	58052

COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH			
c (m)	0.12		Cm	1.0	Fy (kN)	6	Cy 0.044

	Ti	C(Ti)	Re	Te	Te/Ti
		UPPER BOUND			
1	0.588	0.600	13.663	8.040	13.663
2	4.314	0.073	1.666	0.981	0.227
3	2.647	0.171	3.894	2.291	0.866
4	2.469	0.183	4.158	2.447	0.991
5	2.458	0.184	4.181	2.460	1.001
6	2.459	0.183	4.179	2.459	1.000
7	2.459	0.183	4.179	2.459	1.000
8	2.459	0.183	4.179	2.459	1.000
9	2.459	0.183	4.179	2.459	1.000

SEISMIC DISPLACEMENT		TOP WALL DISPLACEMENT	
ζ upper bound (mm)	276	ζ upper bound (mm)	331
ζ lower bound (mm)	0	ζ lower bound (mm)	0

DUCTILITY FACTOR		DYNAMIC AMPLIFICATION EFFECT					
DF =	13.7	Avn	0.15	min limit	3.05	max lim	3.5

TORSIONAL INCREASE IN DISPLACEMENT

not including in this spreadsheet

ASSESSMENT OF PERFORMANCE							
ζ_{kdm}	1.2	drift upper limit(%)	3.513	NG	drift lower limit (%)	0.0	OK
Therefore the foundation width of		1.8	is satisfactory provided the walls design shears are satisfactory				
Vx (kN)	19	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag				85	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT							

Short wall medium soil

WALL GEOMETRY									
H (m)	11.30	Co	1.2	B (m)	0.28	Ve (kN)	7		
L (m)	0.95	V_E (kN)	91	Selected	1.8		T upper limit (sec)		0.707
W_D (kN)	144	V_R (kN)	6	DF	14.8				
Qc (kPa)	550	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.300		G =	60000	Kend	630462			
L2 (m)	0.058		v =	0.35	Kmid	67385			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.150	-0.325	189138	68	10	19978			
2	0.329	-0.146	3931	1	0	84			
3	0.388	-0.088	3931	1	1	30			
4	0.446	-0.029	3931	1	1	3			
5	0.504	0.029	3931	1	1	3			
6	0.563	0.088	3931	1	1	30			
7	0.621	0.146	3931	1	1	84			
8	0.800	0.325	189138	68	54	19978			
		0	401862	144	69	40190			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	0.15			Cm	1.0	Fy (kN)	6	Cy	0.043

	UPPER BOUND				
	Ti	C(Ti)	Re	Te	Te/Ti
1	0.707	0.527	12.342	8.729	12.342
2	4.718	0.067	1.559	1.103	0.234
3	2.910	0.155	3.633	2.570	0.883
4	2.740	0.165	3.872	2.739	1.000
5	2.739	0.165	3.873	2.739	1.000
6	2.739	0.165	3.873	2.739	1.000
7	2.739	0.165	3.873	2.739	1.000
8	2.739	0.165	3.873	2.739	1.000
9	2.739	0.165	3.873	2.739	1.000

SEISMIC DISPLACEMENT		TOP WALL DISPLACEMENT	
| upper bound (mm)	308	| upper bound (mm)	370
| lower bound (mm)	0	| lower bound (mm)	0

DUCTILITY FACTOR		DYNAMIC AMPLIFICATION EFFECT					
DF =	12.3	Avn	0.15	min limit	2.85	max lim	3.5

TORSIONAL INCREASE IN DISPLACEMENT									
not including in this spreadsheet									
ASSESSMENT OF PERFORMANCE									
| kdm	1.2	drift upper limit(%)		3.931	NG	drift lower limit (%)		0.0	OK
Therefore the foundation width of			1.8	is satisfactory provided the walls design shears are satisfactory					
Vx (kN)	18	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag						77	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT									

Short wall soft soil

WALL GEOMETRY									
H (m)	11.30	Co	1.2	B (m)	0.34	Ve (kN)	7		
L (m)	0.95	V_E (kN)	91	Selected	1.8		T upper limit (sec)		0.899
W_D (kN)	144	V_R (kN)	6	DF	15.4				
Qc (kPa)	450	wall thickness		0.3					
srping No.	8		UPPER BOUND						
L1 (m)	0.300		G =	40000	Kend	390286			
L2 (m)	0.058		v =	0.3	Kmid	41714			
node	xi	xi-x (m)	ki	Wi (kN)	Wixi (kNm)	$k(xi-x)^2$			
1	0.150	-0.325	117086	68	10	12367			
2	0.329	-0.146	2433	1	0	52			
3	0.388	-0.088	2433	1	1	19			
4	0.446	-0.029	2433	1	1	2			
5	0.504	0.029	2433	1	1	2			
6	0.563	0.088	2433	1	1	19			
7	0.621	0.146	2433	1	1	52			
8	0.800	0.325	117086	68	54	12367			
		0	248771	144	69	24879			
COMPRESSION BLOCK SIZE				WALL ROCKING STRENGTH					
c (m)	0.18			Cm	1.0	Fy (kN)	6	Cy	0.041

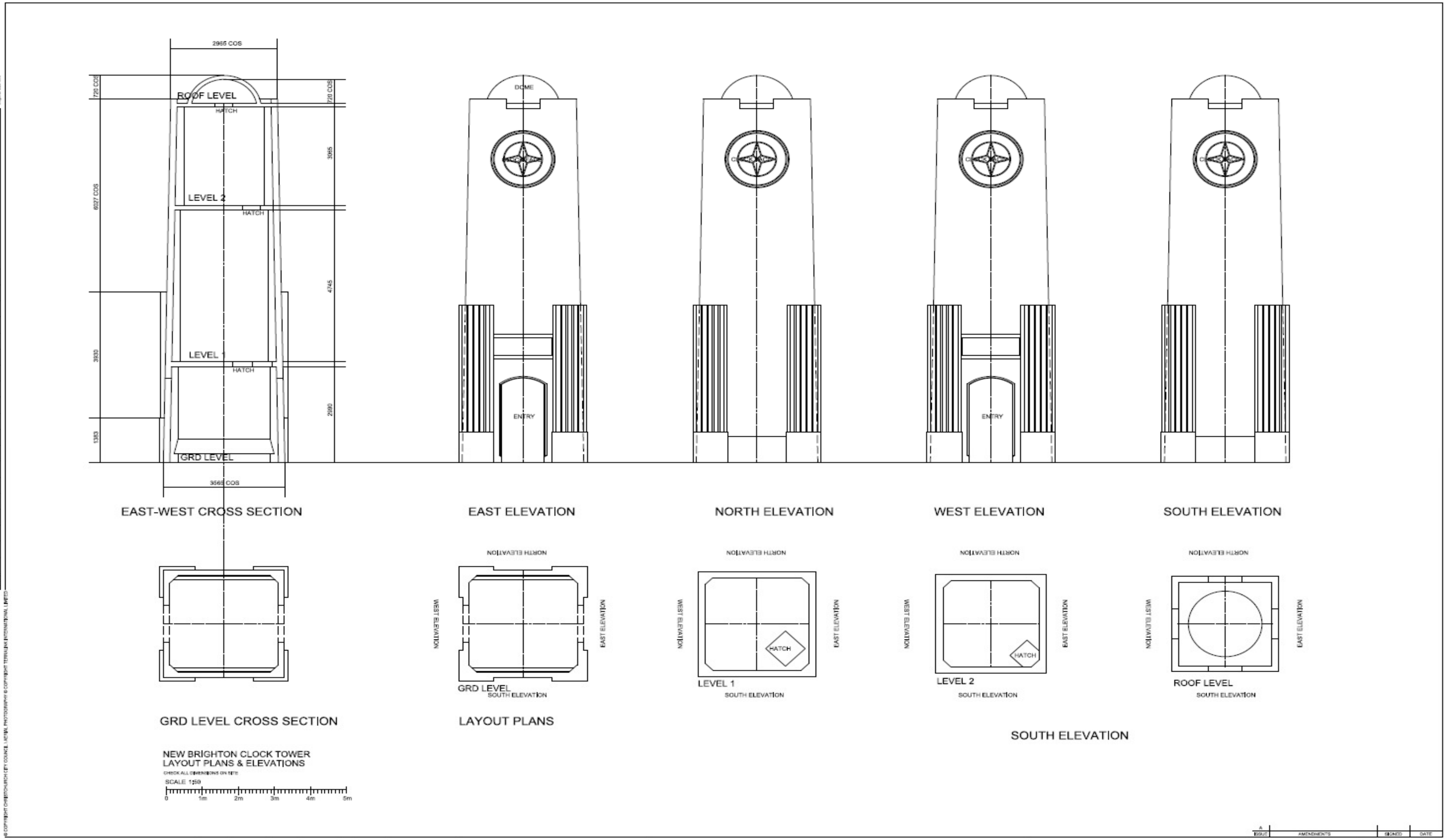
	UPPER BOUND				
	Ti	C(Ti)	Re	Te	Te/Ti
1	0.899	0.440	10.736	9.651	10.736
2	5.275	0.067	1.624	1.460	0.277
3	3.367	0.121	2.942	2.645	0.785
4	3.006	0.149	3.643	3.275	1.089
5	3.140	0.139	3.383	3.041	0.968
6	3.091	0.143	3.479	3.128	1.012
7	3.109	0.141	3.443	3.095	0.996
8	3.102	0.142	3.457	3.107	1.002
9	3.105	0.141	3.452	3.103	0.999

SEISMIC DISPLACEMENT		TOP WALL DISPLACEMENT	
ζ upper bound (mm)	339	ζ upper bound (mm)	407
ζ lower bound (mm)	0	ζ lower bound (mm)	0

DUCTILITY FACTOR		DYNAMIC AMPLIFICATION EFFECT					
DF =	10.7	Avn	0.15	min limit	2.61	max lim	3.5

TORSIONAL INCREASE IN DISPLACEMENT									
not including in this spreadsheet									
ASSESSMENT OF PERFORMANCE									
ζkdm	1.2	drift upper limit(%)		4.318	NG	drift lower limit (%)		0.0	OK
Therefore the foundation width of			1.8	is satisfactory provided the walls design shears are satisfactory					
Vx (kN)	15	corresponds to an aprox shear stress based on shear area of WEB wall of 0.8Ag						68	Mpa
PLEASE CHECK SHEAR STRESS WITHIN CAPACITY OF A CONCRETE WALL WITH REINFORCEMENT									

18 APPENDIX C DRAWINGS



Christchurch City Council
CAPITAL PROGRAMME GROUP

			NAME	SIGNED	DATE
DATUM	C.V.D.				
BENCH MK.		DESIGNED	Design Name		
RL		DES. REVIEW	Review Name		
SURVEY		DRAWN	MJOTT		MAR 2012
SURVEY LB		DRW. CHECK	Check Name		
		FILE LOCATION			
DRAW. REF.					
SAP WBS	721/120/8/11/11	PRINTED ON 27/03/2012 BY			

APPROVED
FOR TENDER
DATE SIGNED
FOR CONSTRUCTION
DATE SIGNED

CONSULTANT
CONSULTANT PROJECT REF.
CONSULTANT FILE REF.

PROJECT TITLE
NEW BRIGHTON CLOCK TOWER

DRAWING TITLE
PLAN & ELEVATIONS

CONTRACT NUMBER ?
ORIGINAL SHEET SIZE A1
SCALES 1:50
CAD DRAWING FILE REF. AD157101
CPG PROJECT FILE NUMBER CP501638
SHEET 1 OF 1